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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

ENGINEERING ECONOMICS AND PUBLIC WORKS
A SYMPOSIUM

NOTE.—Presented at the meeting of the Engineering-Economics and Finance Division, Boston, Mass., October 7, 1937. Written comments are invited for immediate publication; to ensure publication, the last discussion should be submitted by **June 15, 1938.**

ADVANTAGES OF ORDERLY PLANNING

BY FREDERIC H. FAY,[1] M. AM. SOC. C. E.

SYNOPSIS

Attention has been focused upon public works as a possible stabilizing factor on employment, by the vast sums spent by the Federal Government in this field in the past few years for unemployment relief. The paper presents facts as to the construction industry as a whole, its relation to national income, and the share of public works in the industry, for the years 1925 to 1935, inclusive, the period for which detailed statistics were available. For public works themselves, analysis is made of expenditures in this period by the Federal Government, by the States, and by local governmental units.

During the depression expenditures in the construction industry as a whole (which in the pre-depression period accounted for one-sixth of the national income) dropped far more rapidly than the national income, primarily because of the great slump in private construction. Public works declined considerably after 1930, notwithstanding the greatly increased expenditures of the Federal Government, since the excess of Federal expenditures was nullified and much more than offset by the sharp drop in expenditures by counties and municipalities. Contrary to the general impression that the past few years has been a period of substantially increased activity in public works, the facts are that the country's public works program was a full year behind schedule by the end of 1935.

The theory of a controlled public works policy, by which construction will be retarded to some extent in prosperous times and expanded to relieve unemployment in periods of depression, is noted. This principle has not yet been given a fair trial since what has been done thus far has been done under pressure of an emergency and without the benefit of advance planning. The recent depression has taught its lessons, however, which may serve to guide future policy. Some of these lessons are stated. The importance of advance planning of public works and long-term budgeting, whether or not a policy of controlled public works as a stabilizing factor is to be attempted, is reviewed. Suggestions are offered whereby orderly advance planning may be made effective, thereby promoting efficiency in administration and economy in government.

INTRODUCTION

The efforts of the Federal Government during the past five years or more to stimulate public works as a means of relieving unemployment and lessening

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the severity of the depression throughout the United States, has forcibly impressed upon engineers engaged in carrying out this program, and to some slight extent has awakened the public to, the need of orderly advance planning of public works. Although long advocated by a few economists, engineers, planners, and others of vision, it is only during the recent depression that an attempt has been made in this country on a large scale to stimulate business and alleviate unemployment by accelerating and expanding public works construction.

Although it has been measurably successful, this attempt has failed to produce the full degree of stimulation anticipated. However, the experience of the five years (1932 to 1937) affords no adequate criteria for passing judgment on the merits and defects of the principle of planned flexibility in public works, for the reason that (with certain exceptions as regards Federal programs) the element of advance planning has been absent. In State, county, and municipal public works particularly, which constitute by far the greater part of the public works field, carefully prepared plans were almost non-existent. Instead, in a desperate effort to meet an unexpected emergency, project after project was authorized and constructed by means of hastily conceived, unstudied plans which all too often had to be improvised almost over night. The many obstacles in the way of initiating public works at short notice in the absence of carefully prepared plans presented almost insuperable difficulties.

On the engineering side, this lack of thorough preliminary investigation and planning has frequently resulted in uneconomical expenditure through modifications of plans necessary to meet unforeseen conditions arising during construction, the resulting extra cost of which might have been avoided had it been possible to exercise careful engineering foresight based on adequate data as to physical conditions to be met. On the economic side (and this often was of even greater importance) insistence on immediate action left no time for deliberate investigation of the economic worth of many projects undertaken, nor a wise selection between projects according to their comparative merits and need.

The orderly planning of public works in advance of public requirements, both as to their engineering and their economic aspects, is at all times a sound governmental policy. This is so fundamental as to require no argument. That such sensible foresight has not been exercised in greater degree in the past is due largely to lack of understanding by the public generally as to the nature and importance of planning. On the physical side there is little comprehension by the public of the time, effort, and money necessarily required for securing adequate knowledge of site conditions and for the preparation of designs which shall meet conditions in a proper and economical manner. Such engineering investigation, instead of being needless expense, may save far more than its cost by economies effected in construction. On the economic side, there is too often but little attempt, not only to weigh projects as to their need and comparative worth; but also from the standpoint of ability to finance. Contrary to popular impression, there are distinct limits to the funds that can be drawn from the public purse.

Whether or not one holds that there should be some control over public works as a stabilizing factor on employment, the fact remains that advance planning of public works and long-range budgeting are necessary elements of wise governmental economy. They are but the application of the foresight and the common-sense practices of private business, to the business of Government.

On the Engineering Profession, more than on any other group, falls the important duty of guiding and educating the public to the need of orderly planning of public works. It is a highly important part of engineering-economics.

In his address on "The Engineering Profession," delivered at the Commencement Day exercises of the Massachusetts Institute of Technology, on June 8, 1937, Gano Dunn, M. Am. Soc. C. E., laid emphasis upon the responsibilities of the engineer in the field of economics. In discussing science and engineering he stated:

"The essential difference between the scientist and the engineer lies in the economic involvement of the engineer's work. It is in this hot crucible of the economic test that all an engineer does must be tried * * * . Economics is not a physical science. It is a social science, bringing in problems of the human spirit and the behavior of man as man, and it is because the engineer's art deals with dollars and economic relations that he is bound into the great business structure of society in a way that the scientist is not. Being bound into this structure, he must be a man among men. He must be able to make his views prevail. He must be able to persuade and contend. And he must be able to give blows and to take them."

The Construction Industry of the United States

Statistics of the construction industry of the United States have not been available in comprehensive detailed form until recently. The information at hand is that of estimates prepared after long and arduous analysis of statistics of many territorial divisions of the construction field and of many classes of construction. Where these statistics do not cover the entire country estimates must be made of the construction in the territory omitted. A chief source of information is the F. W. Dodge Corporation, of New York, N. Y., which, since 1924, has reported construction contracts awarded in 37 States east of the Rocky Mountains. Another is the volume of building permits issued in more than 800 cities throughout the country, compiled by the United States Department of Labor. Analysis must be made of the construction work done by the many Federal departments, bureaus, and relief and other agencies, supplemented by analysis of construction done by State, county, and municipal agencies. Allowance must be made for construction and maintenance work, by force account labor and not by contract. Duplications must be eliminated. Then, too, it should be recognized that the aggregate totals of contracts awarded and of building permits issued may generally be less than the ultimate cost of the projects which they represent. Innumerable difficulties beset the path of the statistician who seeks to determine the true financial picture of the construction industry.

From its establishment in 1931, the Federal Employment Stabilization Board did notable work in statistical analysis of the national construction industry; this having been undertaken by Corrington Gill, Chief Economist, in co-operation with the Board's executive head, Donald H. Sawyer, M. Am. Soc. C. E. The reader interested in the subject may well consult the writings of Arthur D. Gayer.[2] Estimates of total construction in the United States, based chiefly on the volume of construction materials produced each year, have been prepared by Simon Kuznets, of the National Bureau of Economic Research.[3] The latest publication on this subject that has come to the writer's attention is a *Bulletin* entitled, "The Construction Industry," issued by the Bureau of Foreign and Domestic Commerce of the U. S. Department of Commerce, in April, 1936.

Total Construction, Public and Private.—A comparison of several principal estimates of the total volume of construction in the United States is presented in Table 1(*a*).

TABLE 1.—COMPARISON OF CONSTRUCTION WITH NATIONAL INCOME, IN MILLIONS

Year	(*a*) COMPARATIVE ESTIMATES, TOTAL CONSTRUCTION IN THE UNITED STATES					(*b*) ESTIMATES OF NATIONAL INCOME PRODUCED*		
	Gill (original estimate) (1)	Gill (revised by Gayer) (2)	Gayer (3)	Kuznets (4)	U. S. Department of Commerce (5)	Brookmire (6)	Brookings Institution (7)	U. S. Department of Commerce (8)
1925	$10 805	$11 768	$12 295	$14 032	$11 550	$67 952	$75 918	†
1926	10 912	11 884	12 578	14 343	12 040	71 899	77 177	†
1927	11 153	12 147	12 924	14 876	12 050	71 703	77 003	†
1928	11 339	12 349	13 019	15 919	12 230	76 414	79 679	†
1929	10 492	11 427	12 279	14 381	11 630	79 886	81 940	$80 757
1930	10 108	10 108	10 208	11 921	10 180	71 208	....	67 969
1931	7 586	7 586	7 592	8 920	7 580	59 186	....	53 584
1932	4 361	4 361	4 068	5 458	4 350	45 750	....	39 545
1933	....	....	....	....	3 580	44 681	....	41 813
1934	....	....	....	....	4 440	51 667	....	49 575
1935	....	....	....	....	....	54 826	....	54 955
1936	....	....	....	....	....	....	....	63 799

* See foot-note references to Table 2. † No estimate.

Estimates of the Federal Employment Stabilization Board, as compiled by Gill,[4] are listed in Column (1), Table 1. For the years, 1925 to 1929, values for pipe-line, gas, telegraph, and water-works companies are omitted; but they are included in the years, 1930 to 1932. In Column (2), Table 1, Gayer [5] has corrected the values in Column (1) to include the estimates for pipe-line, gas, telegraph, and water-works construction in 1925 to 1929, by an average ratio, 8.91%, which these items bore to the other construction in 1930 to 1932. Estimates prepared by Gayer, based on data released by the F. W. Dodge Corporation, the Federal Employment Stabilization Board, and the U. S.

[2] "Public Works in Prosperity and Depression," by Arthur D. Gayer, Rept. to the National Planning Board, National Bureau of Economic Research, 1935.

[3] "Gross Capital Formation," 1919–33, by Simon Kuznets (November 15, 1934), *Bulletin No. 52*, National Bureau of Economic Research.

[4] "Public Works in Prosperity and Depression," by A. D. Gayer, p. 65, 1935.

[5] *Loc. cit.*, pp. 65, 69, and 70.

Bureau of Public Roads, are given in Column (3).[6] Kuznets (see Column (4), Table 1) bases his estimates mostly on the volume of construction materials produced each year.[3] These estimates are distinctly higher than the others and are believed by the writer to be in excess of actual amounts. The estimates of the Department of Commerce (Column (5), Table 1) are the most recent, and presumably have been presented with previous estimates in mind. It will be noted that in the 5-yr period, 1925 to 1929, inclusive, the total volume of construction, public and private, averaged approximately $12 000 000 000 annually.

Total Construction in Relation to National Income.—The importance of the construction industry in the national economic structure may be gaged by comparing its amount with that of national income. For this purpose estimates of national income produced are given in Table 1(*b*). In the case of national income, as in total construction, authorities differ in their estimates. Comparison of Table 1(*a*) and Table 1(*b*), however, indicates that in the pre-depression years, 1925 to 1929, inclusive, the total volume of expenditures for construction was roughly one-sixth of the national income. In 1933, however, when construction had reached its lowest level, total construction as estimated by the Department of Commerce (3.5 billion dollars) was only about one-twelfth of the national income produced that year.

Analysis of Estimates of Total Construction.—Beginning with 1925, estimates of total construction in the United States from three sources are available in such detail as to permit a comparative analysis under the classifications of: (*a*) Private construction; (*b*) public utilities (including railroads); and (*c*) public works. Such analysis is given in Table 2. Of the three groups of estimates in Table 2, the first in point of time is that of the Federal Employment Stabilization Board made by Gill. Next followed the estimates of Gayer prepared in 1934 for the National Planning Board and, after some revision, published in 1935 by the National Bureau of Economic Research under the title, "Public Works in Prosperity and Depression." Data for both the Gill and Gayer estimates in Table 2 have been taken from this book. The latest available estimates are those of the Bureau of Foreign and Domestic Commerce of the U. S. Department of Commerce and published in 1936 in its *Bulletin*, "The Construction Industry," from which the data of the third group of estimates in Table 2 are derived. Attention is called particularly to the fact that the estimates in Table 2(*a*) for the years, 1925 to 1929, inclusive, do not include, in the utilities group, expenditures for pipe-line, gas, telegraph, and waterworks construction; hence, for these years, the totals for both utilities and total construction are below their true values. The estimates in Table 2(*b*) and Table 2(*c*) include allowances for construction by these several public utilities in the years named. Beginning with 1930 the construction expenditures of these utilities have been reported.

It will be noted that the public works estimates in Table 2(*b*) are generally higher than those given in Table 2(*a*). From reading the report by Gayer[2] one gains the impression that he makes a good case for the authenticity of his own estimates, and, at the same time, one is impressed by the difficulties and

[6] "Public Works in Prosperity and Depression," by A. D. Gayer, p. 62.

complexities which confront the statistician working in this field. The U. S. Department of Commerce, in its later publication, appears to have favored the estimates by Gill rather than those by Gayer.

TABLE 2.—COMPARATIVE ANALYSIS OF ESTIMATES OF TOTAL CONSTRUCTION IN THE UNITED STATES, IN MILLIONS OF DOLLARS

	1925	1926	1927	1928	1929	1930	1931	1932	1933	1934	1935
(*a*) ESTIMATES BY CORRINGTON GILL, FEDERAL EMPLOYMENT STABILIZATION BOARD*											
Private	5 237	5 365	5 175	5 416	4 451	2 746	1 761	667	....	....	....
Utilities†	2 851	2 935	2 933	2 900	3 265	4 062	2 898	1 629	....	....	....
Public works	2 717	2 612	3 045	3 023	2 776	3 300	2 927	2 065	....	....	....
Total†	10 805	10 912	11 153	11 339	10 492	10 108	7 586	4 361	....	....	....
(*b*) ESTIMATES BY ARTHUR D. GAYER‡											
Private	5 919	5 935	5 552	5 763	4 643	2 558	1 652	450	....	....	....
Utilities	3 564	3 669	3 666	3 625	4 081	4 018	2 873	1 614	....	....	....
Public works	2 812	2 974	3 706	3 631	3 555	3 632	3 067	2 004	....	....	....
Total	12 295	12 578	12 924	13 019	12 279	10 208	7 592	4 068	....	....	....
(*c*) ESTIMATES BY THE U. S. DEPARTMENT OF COMMERCE§											
Private	5 240	5 630	5 240	5 490	4 610	2 800	1 810	680	750	870	‖
Utilities	3 600	3 780	3 770	3 740	4 260	4 150	2 950	1 660	1 240	1 440	1 480
Public works	2 710	2 630	3 040	3 000	2 760	3 230	2 820	2 010	1 590	2 130	2 030
Total	11 550	12 040	12 050	12 230	11 630	10 180	7 580	4 350	3 580	4 440	‖

* "Public Works in Prosperity and Depression."

† Estimates incomplete for years, 1925 to 1929, inclusive, due to omission of values for pipe-line, gas, telegraph, and water-works companies.

‡ Prepared for National Planning Board, published by National Bureau of Economic Research in "Public Works in Prosperity and Depression," 1935. (Compilation by writer.)

§ "The Construction Industry," pub. by Bureau of Foreign and Domestic Commerce, U. S. Department of Commerce, 1936.

‖ Estimate for 1935 is preliminary and incomplete.

PUBLIC WORKS

The distribution of public works expenditures between the Federal Government, the States, and political subdivisions of States, has been investigated by the Federal Employment Stabilization Board and subsequently by the U. S. Department of Commerce. The estimates of these two agencies are in close agreement from 1925 to 1929, after which there is some variation in the item of Federal expenditures. Although in both cases the estimated total volume of public works in each year prior to 1932 is less than that in the Gayer estimates prepared for the National Planning Board (as will be noted from Table 2), the estimates of the Department of Commerce are used herein as the latest and most complete breakdown of expenditures for public works. These data of the distribution of public works expenditures between the Federal Government, States, counties, and cities, for the years, 1925 to 1935, inclusive, are given in Table 3. In this table, Federal aid is included in Federal expenditures and excluded from those of States and their political subdivisions.

TABLE 3.—DISTRIBUTION OF PUBLIC WORKS EXPENDITURES
(Estimated* by Department of Commerce; Federal Aid Is Included in Federal Expenditures and Excluded from Other Divisions)

Year	(a) DOLLAR DISTRIBUTION, IN MILLIONS					(b) PERCENTAGE DISTRIBUTION				
	Cities	Counties	States	Federal	Total	Cities	Counties	States	Federal	Total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1925	1 280	780	410	240	2 710	47	29	15	9	100
1926	1 300	680	400	250	2 630	49	26	15	10	100
1927	1 480	880	440	240	3 040	49	29	14	8	100
1928	1 420	830	500	250	3 000	47	28	17	8	100
1929	1 340	560	580	280	2 760	49	20	21	10	100
1930	1 500	710	710	310	3 230	46	22	22	10	100
1931	1 300	330	780	410	2 820	46	12	28	14	100
1932	800	140	550	520	2 010	40	7	27	26	100
1933	400	100	500	590	1 590	25	6	32	37	100
1934	500	130	400	1 100	2 130	23	6	19	52	100
1935	550	130	350	1 000	2 030	27	6	17	50	100

* "The Construction Industry," 1936, Bureau of Foreign and Domestic Commerce, U. S. Dept. of Commerce.

In the pre-depression years, 1925 to 1929, inclusive, the average public works expenditure in each of these four divisions was as follows:

Division	Expenditure	Percentage of total
Cities	$1 364 000 000	48.2
Counties	746 000 000	26.4
States	466 000 000	16.5
Federal	252 000 000	8.9
Total	$2 828 000 000	100.0

Thus, in this pre-depression period, local governmental units accounted for three-quarters of the total expenditures for public works, and the State Governments for one-sixth, whereas the expenditures of the Federal Government were only about 9% of the total. Analysis of Table 3 reveals some striking facts as to what actually happened to the public works program during the depression.

Federal expenditures which, prior to the depression had averaged about 250 million dollars annually, rose steadily during the Hoover Administration and reached 590 million dollars in 1933. The Roosevelt Administration program inaugurated in 1933, increased them to one billion or more in 1934 and 1935. The States themselves expanded their construction activities from 1929 and carried forward enlarged programs until 1933, after which year State expenditures dropped somewhat below the pre-depression average. City and county expenditures, however (which together averaged 2 110 million dollars prior to 1930 and amounted to 2 210 millions in the latter year), dropped sharply to 500 millions, or less than one-quarter of the former values, in 1933, and increased but slightly in the two years following.

Considering the 6-yr depression period, 1930 to 1935, as a whole (6 yr being taken as 1935 is the latest year for which data are available), and contrasting it with the pre-depression period, 1925 to 1929, it is found that, in the aggregate,

city and county expenditures were more than 6 billion dollars less than they would have been had the pre-depression average been maintained. On the other hand, the aggregate of State expenditures was nearly half a billion, and of Federal expenditures 2.4 billion, more than the maintained pre-depression figures. The net result is an aggregate shrinkage of more than 3 billion dollars in public works expenditures from 1930 to 1935, inclusive. In other words, contrary to the general impression that these depression years were years of stimulated activity in public works, the facts are that the country's public works program was a full year behind schedule at the end of 1935.

Federal vs. Local Public Works.—A paper entitled "Federal *vs.* Local Public Works," was presented in 1937 by William Stanley Parker,[7] based on the data reproduced in Table 3. Mr. Parker's conclusions as to the "depression relationship" between Federal public works and those of the combined State, county, and city groups (which he classifies, collectively, as "local" public works) are so pertinent that all interested students of the subject will wish to read it at the source.

According to Mr. Parker, the 1929 commitments for "local" public works formed the peak in 1930. The beginning of the slump began to show in 1931, and the decline in local expenditures than continued sharply to a low point in 1933. Federal expenditures, on the other hand, began to increase in 1929; in 1932 they had doubled, and had increased still more in 1933. Quoting Mr. Parker: "These increases represent the effort of the Hoover administration to offset the mounting unemployment in the construction industry." The comparison of the cumulative totals for the 3-yr period, 1931, 1932, and 1933, is as follows:

Decrease in "local" expenditures (below the 1930 peak)	\$3 800 000 000
Increase in Federal expenditures (including the 1929 and 1930 increases)	\$ 860 000 000

An accumulated shortage of about 3 billion dollars is indicated by the foregoing data, or the equivalent of an entire "normal" year's work. The efforts of the Federal Government are thus seen to be relatively ineffective.

The new Federal relief program was initiated in 1933 and developed in subsequent years. Expenditures on Federal projects were increased, "grants were made to local and state authorities and loans at low interest rates sought further to stimulate local expenditures." At the same time, Mr. Parker states that the Federal Government was spending large sums for relief, a substantial proportion being on minor construction projects which were not included in his basic statistics. Quoting:

"Federal expenditures on public works, including grants, mounted to \$1 100 000 000 in 1934 and to an estimated billion in 1935, four times their 'normal' amount. What happened to local expenditures in these two years? The expenditures of states dropped steadily, from the 1933 total of 500 million to 400 million in 1934 and 350 million in 1935. County expenditures increased about 30 million and cities increased theirs about 150 million, the net result being that in 1934 and 1935, in spite of the urgent need of employment, especi-

[7] *The Architectural Record*, May 1937, p. 12.

ally in the construction industry, in spite of the efforts of the Federal government to incite local governments to develop their programs, and in spite of the local help through Federal relief expenditures, the combined expenditures of states, counties and cities for public works remained practically the same through 1934 and '35 as they were in the low peak year of 1933, being only little more than one-third of their 'normal' amount."

Furthermore, Mr. Parker stresses the fact that statistics for 1935 reveal that the total bonded indebtedness of States, counties, cities, and towns was 350 million dollars less than in 1932, indicating "a strange failure of local governments to cooperate with the Federal program which had come to their rescue."

His comparison of cumulative totals for 1934 and 1935, similar to the 3-yr period, 1931, 1932, and 1933, is as follows:

Decrease in local expenditures (1934 and 1935)	$3 780 000 000
Increase in Federal expenditures (1934 and 1935)	1 600 000 000

Comparing the new annual totals in these two categories:

Federal expenditures	$1 000 000 000
All local expenditures	1 000 000 000

and, as a final summary, for the years since 1930:

Cumulative decrease in local expenditures	$7 580 000 000
Cumulative increase in Federal expenditures	2 460 000 000

In other words (to quote Mr. Parker), "for every dollar the Federal government increased its expenditures in order to prime the pump the local governments reduced their expenditures by three dollars, and even after the 1933 relief program developed the local reduction was more than twice the Federal increase."

Planning and Control of Public Works

Several of the States, notably New Jersey and Idaho in 1915, Pennsylvania in 1917, California in 1921, and Wisconsin in 1923, passed legislation providing for expansion of public works construction as a remedy for unemployment, but these measures either remained inoperative or produced negligible results. A few city and county governments have undertaken to prepare long-term public works programs. Thus far but little has been accomplished in this direction by the States and their political subdivisions.

For a dozen years following the World War, attempts were made repeatedly to secure Federal legislation for the orderly planning and control of Federal public works. These efforts proved fruitless until 1931 when Congress passed legislation creating the Federal Employment Stabilization Board. The purpose of the Act was to provide for "the advance planning and regulated construction of public works, for the stabilization of industry, and for aiding in the prevention of unemployment during periods of business depression."

By the Employment Stabilization Act of 1931 the Federal Government established a policy which, if followed consistently by State and municipal

governments throughout a period of years, may not only assist in stabilizing employment, but at the same time should result in wiser and more economical expenditures for public works. An important feature is the requirement that Federal departments and other agencies having charge of construction shall "prepare a six-year advance plan with estimates showing projects allotted to each year," with the further provision that the 6-yr program be kept up to date by annual revision and extension. Thus, the principle of advance planning has been established by law. A second important feature is recognition of the need of co-operation in advance planning between the Federal Government and State and municipal governments if control over public works is to be in any degree effective. The Federal Employment Stabilization Board was directed to watch, constantly, the pulse of the construction industry, to keep the President informed regarding business activity and the trend of employment, and to give warning of the existence or approach of periods of business depression and unemployment so that at such times the President might advise the Congress regarding needed expansion of the public works program.

In the Federal Employment Stabilization Board the theory of planned public works was put into large-scale operation for the first time in the United States. The Board compiled the first reliable data on construction expenditures in the post-war period. Its work was of great assistance to the Public Works Administration in determining the allocation of funds appropriated under the National Industrial Recovery Act of 1933. The latter Act created the Federal Emergency Administration of Public Works which, in turn, set up the National Planning Board as its advisory body. Both these agencies took over some of the functions of the Stabilization Board which then continued to operate as the Federal Employment Stabilization Office of the Department of Commerce.

Basic Principles.—A controlled public works policy is usually looked upon as a flexible policy which will exert some degree of stabilizing effect on changing business cycles—a policy by which public works construction will be retarded to some extent in prosperous periods and expanded to relieve unemployment in periods of depression. The principle of controlled public works has not yet been given a fair trial since what has been done thus far has been done under the pressure of an emergency and without careful advance planning. It has not yet been put into large-scale operation in the United States in prosperous times and the extent to which it may lessen the severity of a depression has not been demonstrated. The recent depression has taught its lessons, however, which may well serve as a guide to future policy. Among them may be noted the following:

(1) Long-range advance planning and budgeting are absolutely indispensable for the success of any plan of controlled public works.

(2) The Federal Government cannot undertake the task alone. The excess of Federal expenditures for public works since 1930 has been nullified and much more than offset by the sharp drop in expenditures by counties and cities. To be successful, State and local governments must do their share. It s a nation-wide problem.

(3) The financial program, or long-range budgeting, is an important element. In prosperous times some cities have wisely followed, to a greater or less extent,

a "pay-as-you-go" policy, paying for capital expenditures out of current income, thus accumulating reserves of borrowing capacity and having strong credit with which to face a depression. This has been true of certain State Governments as well. Recently, the suggestion of creating reserve funds in boom periods to be drawn upon for public works construction in periods of depression has been given publicity. In general, the financing of public works during depressions, especially if construction programs are then to be expanded, may be accomplished in one of two ways; either by drawing upon previously accumulated reserves, or by long-term or short-term borrowing. Of these two methods, borrowing, preferably for short terms, is generally favored. Federal and State credit is usually such that borrowing presents no serious difficulties. This may not be the case with local units on which rigid limits may have been placed as to borrowing capacity. Such restrictions have doubtless accounted, in part, for the sharp drop in city and county public works expenditures since 1930. Long-term budgeting must concern itself with the problem of conserving the borrowing capacity of these local units or of finding means (as has recently been done by legislation in several of the States) whereby restrictive limits on borrowing by local units are raised somewhat during an emergency caused by a depression.

(4) A controlled public works policy should not be looked upon as a means of relieving, directly, unemployment among workers outside the construction industry by giving them employment on construction projects. In any important depression period private construction will surely decline, public utilities may be forced to curtail their construction activities somewhat, and inevitably there will be considerable unemployment among workers normally employed in these branches of the industry. Controlled public works construction, however much it be expanded during a depression, should give direct employment only to those workers within the construction field.

(5) The unemployed must be supported in any event out of public or private funds. The real net cost of public works during a depression, apart from any stimulating effect upon the business cycle, is actually much less than it seems because, in public works, the community gets something in return for its expenditure.

Some may ask, "Can a sufficient volume of genuinely needed public works be found in depression periods to give substantial employment?" The answer is "yes," if construction programs are carefully planned in advance and retarded in periods of prosperity. Roads and bridges are examples of construction suitable for a flexible program.

The construction industry, as a whole (in which, in normal times, expenditures are roughly one-sixth of the national income) is a highly important part of the national economic structure. Public works normally constitute about one-fourth of this industry. Public works alone give primary and secondary employment to a substantial number. In primary employment it has been estimated that normally about 800 000 workers are directly employed in the field, and for every worker directly employed one or two others are employed indirectly in the manufacture and transportation of construction

materials. Thus, the normal primary employment due to public works may be, roughly, 2 000 000 wage earners. These workers, in turn, spend a very large proportion of their wages for consumer goods and services thus creating a "secondary" employment in other fields perhaps, roughly, as large again.

If it were possible to maintain the construction industry as a whole at a fairly uniform level, much would be accomplished in "flattening" the business cycle curve and in effecting a marked stabilizing influence on national income. The volume of private construction, however, is not susceptible to public control and is certain to fluctuate widely. The control of public works construction such that its volume remained fairly constant, or was expanded somewhat in depressions, would have appreciable effect in lessening the severity of a depression in the construction industry and, at the same time, would stimulate employment, to some extent, outside that field. If public utilities should join with public works in stabilizing their construction programs, these two (which, together, account for more than one-half of all construction activity of the country, because of the increased primary and secondary employment thus created), would measurably improve the business of the country in depression periods. Such co-operation is not possible at present, however, under existing taxation laws.

Federal Aid in Non-Federal Construction Work

During the recent depression, the Federal Government has made a radical departure from principles to which it had adhered for a century and a half by embarking upon the new policy of lending its credit, and of giving grants outright, to aid the construction of projects which it does not own. That this is an emergency policy, of temporary nature, which will shortly be brought to an end, is generally understood. The precedent having been established, however, undoubtedly there will be an insistent demand for the resumption of this policy when the next severe depression comes. If that is the case, long-range advance planning becomes vitally necessary.

It was in 1932 that the United States Government inaugurated the policy of using its credit to aid borrowing by local governments for their public works. By an Act passed in July of that year, Congress authorized the Reconstruction Finance Corporation to make loans, up to the aggregate amount of $1 500 000 000, to States and their political subdivisions and agencies, and in certain instances to private corporations, for "self-liquidating" construction projects. Although this was the first time that Federal credit had been used in this country for construction of other than Federal public works, the idea was not new. For forty-five years this policy had been at work in England where, through the agency of the Public Works Loans Board, most of the funds for housing and similar projects undertaken by the smaller municipalities had been borrowed from the National Treasury.

This 1932 Act was a demonstration of the growing belief in the value of expanded public works programs as a means of affording unemployment relief. Because of the stipulation of the Act that loans by the Reconstruction Finance Corporation must be restricted to revenue-producing or "self-liquidating" projects, and of the policy of the RFC in charging relatively high interest rates,

authorizations were made very slowly and disbursements more slowly still. Only a minute fraction of the one and one-half billion dollars authorized had been put to work for the relief of unemployment before this Act was superseded by that of 1933.

The National Industrial Recovery Act of June, 1933, appropriated $3 300 000 000 for public works; authorized grants as well as loans to States, municipalities, and other public bodies; created the Federal Emergency Administration of Public Works (PWA) to administer the Act; and required the PWA to "prepare a comprehensive program of public works," Federal, State, and local.

Engineers throughout the country, perhaps better than any other group, realize the chaotic conditions as regards public works planning precipitated by the 1933 Act. In the feverish haste to put the vast Federal appropriation to work for the relief of unemployment there was no time to prepare orderly programs or even adequate preliminary plans. Almost over night as it were, engineers everywhere were called upon to prepare applications for public works projects in the form required by the PWA, which all too often involved guesses as to site conditions, estimates which were scarcely more than guesses of probable cost, and plans which were the merest sketches. There was no time for economic planning as to the need and comparative worth of various projects. Thus, at the outset and for a long time after the 1933 Act became operative, orderly planning was not possible and scant attention was paid to a "comprehensive program of public works."

Under this Act, however, the Administration committed itself to the principle that orderly planning and control was necessary to secure efficient and socially desirable expenditure of public monies in the public works field. The Act contained no statement concerning the possible utilization of public works as a stabilizing factor, but the prominence, in the Act, of the stipulation regarding the formulation of a comprehensive, nation-wide program seemed to imply that in the future such a program could be utilized toward that end.

Within the framework of its initial organization the PWA set up in 1933 the National Planning Board, an advisory body, to which was assigned the preparation of the comprehensive program required by the 1933 Act. The functions of the National Planning Board, as stated, were to engage in research concerning the potential development of public works in relation to social and economic needs and to draw up a comprehensive plan for regional areas in co-operation with National, State, and local agencies. As for planning agencies throughout the country, in the summer of 1933 there existed at the top of the pyramid the newly created National Planning Board and at its base about 700 municipal planning boards in the principal cities and in certain of the larger towns. Between the top and the base, State and regional planning agencies were almost wholly non-existent.

In that year the President addressed communiciations to the Governors of the several States urging the creation of State planning boards to assist in the orderly planning and programming of public works in their respective States, particularly as an aid to the PWA in the allotment of grants and loans. Applicants to the PWA for such Federal aid were required to state whether the

projects in question formed parts of comprehensive plans and had received the approval of State and local planning agencies.

New Hampshire was the first State to comply with the President's request and immediately Governor Winant appointed an unofficial State Planning Board. The example set by New Hampshire was followed by a large proportion of the States. In most cases these boards were necessarily unofficial bodies, at first, created by executive order so as to avoid the delay of waiting for State legislation. At present (1937) there are State Planning Boards in forty-five of the forty-eight States, and of these, thirty-six Boards are now permanent official bodies established by legislative act.

The National Planning Board has fostered the creation of interstate regional planning commissions when groups of States have problems of special regional interest. The New England and Pacific Northwest Regional Planning Commissions are examples of such bodies.

The experience of the four years, 1933 to 1937, in attempting to promote useful and needed public works projects as a part of the Federal aid program for unemployment relief, has demonstrated conclusively to those concerned with these public works, if not to the public generally, both the need of advance planning and that of fitting public works plans into a logical pattern. If any lesson is to be learned from the experience of the last few years, it is that public works should no longer be planned and constructed in a "hit-or-miss" fashion but that henceforth, the need for them should be studied with proper foresight. The Roosevelt Administration deserves commendation for its stand regarding the necessity for advance planning, and for its efforts toward co-ordinating public works projects, Federal, State, and local, into something like an orderly program which will promote wise and economical use of public funds in this field.

Advance Planning

Whatever differences of opinion may exist as to the wisdom of some control over public works as a stabilizing factor on employment, all must agree that advance planning and long-range budgeting are common-sense business practices. They are absolutely essential in private business. They are practiced likewise by leading public utilities of which the nation-wide telephone company is a notable example. It is in the business of Government, particularly in the case of public works, that the necessity for foresight and economy is all too often ignored.

Advance physical planning of public works requires some modification of the practices usually obtaining in governmental departments, particularly those of the State and local governments. Rarely do these departments take a long-range view and give thorough and detailed study to their requirements for some years in advance. Usually, the departments themselves are not primarily at fault. Mistaken governmental economy in far too many cases allows such meager appropriations for technical service that thorough engineering investigation of public works projects well in advance of their authorization is not possible. Frequently such projects are authorized on hastily prepared plans and estimates, without careful advance study based on adequate knowledge of conditions to be met. Only when funds for construction become available is it

possible, in many cases, to start a general engineering investigation and by that time pressure to get construction "under way" leaves little time for thorough engineering planning.

The departments themselves may often be subject to criticism, however, for failure to impress upon appropriating agencies the wisdom and ultimate economy of adequate appropriations for engineering service which will provide a competent organization sufficient to handle current work and, at the same time, give proper consideration to work foreseen as needed in the future. Too many governmental departments live on a "hand-to-mouth" or day-by-day policy. At most, it is from administration to administration. A new governor or a mayor is elected; new department heads are appointed. There is usually a sincere intention on the part of the Chief Executive to administer the affairs of the State or city wisely and well, but no carefully prepared and formally accepted long-term program or pattern awaits his coming. There is all too often, therefore, a perfectly understandable human tendency to discount the efforts, the efficiency, and perhaps even the sincerity, of earlier administrations; to abandon projects already begun; to divert—with consequent loss—millions of dollars from one channel of public activity to another; and to venture into new and unexplored fields without the benefit of proper advance technical planning. In the absence of advance planning, under the present system of Government, and in the light of human nature, this cannot be otherwise. No one can change human nature. Americans do not want to depart from their democracy in Government; they can and should prepare well in advance, and within reason should adhere to, carefully considered programs for public works activities.

The public thinks engineering inconsequential and something that can be done in a hurry when required. There is great need of public education as to the importance of engineering investigation before proceeding with construction. The serious lack of adequate advance physical planning by departments of State and local governments has been glaringly evident in connection with the current program of Federal aid in non-Federal public works.

Advance financial planning or long-range budgeting of the public works program is of equal importance with advance physical planning. A few American cities have already led the way in preparing long-time bond or financial programs for public improvements. If the example which they have set were general practice in State and local governments, greater economy and more uniformity in expenditure would result. The danger of rushing ahead with poorly considered and less necessary projects, and with projects which the community can ill afford, would be largely reduced.

Timing of public improvements is an important part of advance planning. Proper timing will result if there is co-ordination of advance physical planning and advance financial planning; in fact, all three are factors of a single problem. Timing so as to retard public works programs somewhat in periods of prosperity and to advance them in depressions is necessary if public works are to be utilized as one phase of an economic stabilization program.

Co-ordinated, long-range public works programs are made up of many diverse elements. They are desirable and economically necessary for city,

county, State, and Federal Governments. How then is proper co-ordination to be secured? It is not a matter that can be handled adequately by individual governmental departments alone. Each department, it is true, is best qualified to forecast and plan for the needs of its own particular field, but its vision is limited, it may know little or nothing of the needs of another department, it is naturally biased to a greater or less extent by its conception of the importance of its own problems, and it is not an impartial judge. In no governmental unit, State or local, will a well-considered and well-co-ordinated long-range program of public improvements, best suited to the needs of that unit as a whole and within its capacity to finance, result from a round-table discussion between representatives of the departments directly concerned with carrying out these improvements. A broader, more detached, and more impartial viewpoint is needed.

Co-ordination is planning in its broadest terms. It can well be secured through some form of non-political planning agency, acting independently of other departments of the Government, free from all administrative responsibility for the construction of projects, and one in which a substantial proportion of its members, at least, shall be appointed in rotation for fairly long terms in order to secure continuity of policy. Such planning agency should work in closest co-operation and harmony with other governmental departments responsible for public improvements and for the Government's financial administration. Heads of certain of these departments may well be members, *ex-officio*, of the central planning agency in order to secure closer co-operation and a better understanding of public works problems and their relation to the Government's financial policy. It would seem best, however, that the majority of the members of the planning body should be free from other governmental duties and responsibilities in order that they may exercise unbiased judgment. Such membership presents an opportunity for engineers, architects, planners, economists, business men, and civic leaders of highest standing to render service of great value to their community.

If the challenge is fairly met, and public officials work in co-operation with the planning group, then an orderly progress may be expected in promoting a long-term program of public works, in establishing efficiency in administration, and in effecting real economy in government.

INFLUENCE OF PUBLIC OPINION

By Daniel W. Mead,[8] Past-President and Hon. M. Am. Soc. C. E.

Synopsis

A well informed public can and will raise such objection to projects which they know to be unsound that no Federal, State, or Municipal Administration will dare to undertake such projects.

Public works are, in general, so complicated as to be little understood by the general public. In recent years and in the past, many uneconomical public works have been undertaken, for reasons of political expediency, and have gone unchanged because the public has not understood their unsound character. This lack of understanding by the public is largely due to the fact that the Government seldom furnishes the information necessary for a basis of sound judgment, but misleads and misinforms the public by extravagant statements and propaganda favorable to such projects.

Engineers, from their training and experience, can and should study these projects and inform the public, in simple language that can be readily understood, the facts concerning such projects that will give the best possible knowledge from which the economic feasibility and desirability of such ventures may be judged. Some of the Government projects of the past, present, and probable future, are discussed briefly, herein, and these objectionable features pointed out.

The Record of Government in Business

There is no profession more affected by depressions than that of Engineering. In consequence, there are no men who are more interested, and none better qualified to assist, in the re-establishment of normal conditions than engineers.

After seven years of waiting (1930 to 1937) which includes five years of Federal Administration experiments, the depression is still present. Whether partial recovery, as far as recovery has occurred, is due to, or in spite of, administrative activities will be a moot question for years to come. There are still under consideration many suggestions for further radical changes in social and business relations.

The engineers of the United States will do less than their duty if they hesitate to discuss these matters frankly, honestly, intelligently, and fearlessly, and bring to bear on the solution of these vital problems the best thoughts of their profession.

Many ill-conceived and unwarranted public and semi-public works have been undertaken by the Federal Administration since the beginning of the

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depression. The more these ill-considered and extravagant projects are discussed and their defects made plain to the public, the more likely it is that they will be prevented in the future by the development of a sound public opinion. To the mind of the writer, a well informed public can raise such objections to unsound projects that no administration—municipal, State, or Federal—will dare attempt projects that are not based on thorough investigation, sound judgment, and sound economies.

The writer believes that there are many matters at the present time that are even more important to the engineer than the very important matter of ill-advised public works. On account of the introduction of labor-saving machinery and other mechanical devices that have saved labor, increased production, and reduced cost, the Administration has charged the Engineering Profession with the production of unemployment. The facts are diametrically opposite to this theory.

The National Resources Committee reports:[9]

> "* * * in the period from 1870 to 1930, the population of the country multiplied by three while the number gainfully employed multiplied by nearly four. * * * The eighteen major industries of today which employ directly and indirectly about 25 per cent of those gainfully employed in this country were absolutely unknown in 1870. While mechanical power has been instrumental in separating the worker from his tools and agriculture from industry, it has definitely created more callings and more employment than it eliminated."

The Administration has also urged the engineering schools to teach their students to consider not only the design and construction of engineering works but also to study and determine their ultimate effects on society.

Can the Administration believe that the recent construction of vast public works, many of which will compete with projects financed through the sale of securities to the public under the supervision of the various State commissions, is fair to such investors, or that the construction of such works will effect recovery, or be a real benefit to the nation? Are not all honest investors in the securities of public utilities entitled to the consideration of the Administration in the planning of public benefits to the citizens of the United States? This question has been raised frequently but the writer has heard no answer.

The Administration's suggestion for careful consideration of the effects of improvements would have been received more sympathetically if the plans for the construction of these great public works undertaken for the alleged purpose of recovery, could stand sound economic analysis.

The entrance of Government into business is being carried so far by the present Administration that it seems to threaten the ultimate socialization of business and industry and the regimentation of all professional men and workers into Government service.

The record of Government in industry, whenever it has been tried, is not a satisfactory one and warrants no further extension of Government into business. The fiascos of the Government control of the railroads during the World War, of the Government-owned railroad in Alaska, the Government ownership of railroads in Canada, seem to have little weight on public or political opinion.

[9] "Report on Technological Trends and National Policy," June, 1937, p. 254.

The economical records of Federal irrigation projects since 1902 and of the Inland Waterways Corporation since its organization, are thoroughly unsatisfactory. The recent records of the Resettlement Administration and of the Federal Housing Projects are still later exhibits of Federal economic ideas. Can the public and its representatives in Congress learn nothing from experience? or is the trouble due to lack of information? The unfortunate results of these disastrous experiments should be brought to the attention of the public so that they will realize the danger of further experimentation in Government ownership and operation.

An immediate danger that confronts the nation is the further entrance of the Government into the hydro-electric power field. Already the Government is committed to vast expenditure for power development, partly by Acts of Congress, but largely by administration orders. Among such ventures may be named the Tennessee Valley Authority (TVA) involving an expenditure of about a half billion dollars, the Grand Coulee Project which will involve a similar sum, and diverse other Federal projects, including the Passamaquoddy Project, now sleeping, but not forgotten.

The bill before Congress (S. 2555), if it becomes a law, will divide the United States into seven districts, in each of which an authority will be established, essentially similar to the TVA. This will practically place all future hydraulic power development under Federal ownership and operation. It will probably involve the Government in the installation and operation of steam or other auxiliary power plants as in the case of the original Muscle Shoals Project. It may also involve not only Federal navigation and flood protection, but other industries, and ultimately may eliminate local, State, and probably all private industries on a theory of "unified" developments.

It seems to the writer that this bill, if passed, and these policies if continued, will involve "Federal engineering," "Federal domination," and will prove a serious detriment to the future, of not only the Engineering Profession, but of all other professions, trades, and industries.

The writer is opposed to these policies as they involve a direct attack on the liberty of the American people, on individual freedom, and on industrial and professional independence.

Destruction of Private Property by Government Ownership

In the early development of the United States water transportation and water power had a great influence on settlement, and on community development. The eastern seaboard was the section of the country first available to settlement, and local water powers first became the centers of industrial development. The extension of settlement developed, normally and naturally, along the navigable waters. In early days, and quite up to the time of the Civil War, traffic was largely water-borne. Many of the early canals and the early river improvements, even while in themselves largely unprofitable, were warranted by the development of the country which resulted therefrom. As the ox-team and the canal-boat were displaced by the railroad and other more rapid means of transportation, internal navigation on rivers and canals has gradually assumed less and less importance; yet the tradition of the great value

of internal navigation has caused the Federal Government to waste hundreds of millions of dollars on navigation projects, such as the Fort Peck Reservoir and the 9-ft channel in the Upper Mississippi River, of little present or prospective economic value.

In the same way the early use of water power has given it a value in the public mind, and more especially to the politician, which in many cases it does not possess. The fact that flowing water is "energy going to waste" is overemphasized and the false value attributed to water power is causing the Federal Government to undertake, and to encourage others to undertake, many developments that are entirely unwarranted, and some of which will probably be utter failures. Such success as is attained will be largely at the expense of private investments already made and will probably involve very great losses to many investors. It required many and severe losses in investments to prove the necessity of limiting competition in the matter of public utilities. In the past, competition was permitted which encouraged unintelligent expenditures and which resulted in heavy losses to competitors, and in entirely unsatisfactory and unduly expensive public service. Every loss to an individual is a loss to the nation. The destruction of private property makes the entire country poorer. It was only after much unfortunate experience that competition was eliminated and regulation substituted with much more satisfactory results.

The Administration, however, has again initiated the uneconomic principle of unrestricted competition, not only by the duplication of plants but also by paralleling transmission lines. Private enterprises suffer in competition with Government ownership because of the unfair advantages the latter has over the former. With capital furnished from taxation and not directly charged against the development, and by omission of taxes, certain Government projects can be made to appear successful and can destroy any private competition.

Indeed many private competitors are being taxed to effect their own destruction. "The right to tax is the right to destroy." The writer believes that intelligent public opinion is in favor of a "square deal" and the securing of satisfactory public service at reasonable prices through intelligent regulation and without uneconomical or unfair Government competition. If Government ownership and operation is other than politically desirable, it should be introduced in a frank, honest, and honorable way.

Government Function in Business Recessions and Recovery

Depressions are not uncommon in this or other countries. Many now living will recall some or all of the depressions of 1884, 1893, 1907, and 1921; and the depressions of 1817, 1827, 1837, 1847, 1858, and 1873 are matters of history.

In all previous depressions, the responsibility for recovery has been met largely by the business interests of the country, and recovery has been brought about within a reasonable length of time. The present is the first time that the Federal Administration has assumed the responsibility of recovery.

Administrative action is apparently based on the theory that recovery can be effected only by an entire re-adjustment of business and social relations under strict Federal laws and regulations, and that radical changes are necessary

in the basic principles of the American Government, which principles have been highly satisfactory during the last one hundred and fifty years.

Long experience has demonstrated that legislation passed to improve social conditions frequently results in effects quite different from what were expected, and usually must repeatedly be changed and modified to accomplish the results desired even approximately. It seems strange, therefore, that apparently both public and political opinions still adhere to the belief that almost any evil in social or business relations can be remedied immediately by "passing a law." Satisfactory social changes are not accomplished so simply.

In spite of the warnings of experience, the Federal Government has been, and still is, passing radical legislation which can but delay recovery on account of the experimental character of the changes involved and the effect of the uncertainties on public confidence. C. F. Roos, Director of Research of the defunct National Recovery Administration (NRA), stated:

"* * * although I was invited to join the NRA staff as a friend of the new administration and went to my work with a sympathetic attitude, the data accumulated by our research workers forced upon me the conviction that their type of legislation was a mistake. * * * These studies also show that NRA regulations of hours and wages kept business recovery at a standstill for a two year period."

To care for the unemployed, the Public Works Administration (PWA) was organized and has been the most satisfactory and effective activity attempted by the Administration. It has been, as far as it has been allowed to act, fully as satisfactory as could be expected from any Federal emergency agency. Securities issued by various municipal organizations and not otherwise salable under the conditions of the depression, were taken by the Federal Government, and the avails plus a bonus of 30% (which was afterward increased to 45%) were furnished under Federal supervision of projects.

The writer believes that the principle of Federal grants was a serious mistake. Such grants must ultimately be paid by the taxpayers of the entire nation and could not be made with any degree of equality throughout the country. The grant encourages extravagance in local public works, not immediately necessary, in order to obtain the subsidy which the locality must pay in part in any event. Works of real necessity would have been assured by Government loans at low rates, which the community benefited would ultimately have been obliged to pay. Such a plan, therefore, would not have resulted in an unfair distribution of benefits and taxes.

The increase in grant from 30% to 45% has resulted in some communities receiving 50% greater grants than other adjoining communities that were more prompt in the efforts to adopt relief work. Other communities that had not sufficient influence to secure such grants were independent enough to pay the entire costs for needed works and will now, by the payment of taxes, unfairly be obliged to help other communities that received Federal assistance.

In the main, the PWA seems to have been well organized and the works well selected and well conducted, when all conditions are considered. In some cases, however, monies were appropriated for the purpose of constructing municipal plants where cities were already served by private corporations.

Power districts were also organized and subsidized in locations adequately served at reasonable rates. In some cases not only were the projects subsidized by furnishing 45% of the cost, but the Administration purchased the bonds issued as no other market for such bonds was available, and thus such questionable projects have been or are being constructed at Federal risk with no other responsible organization behind them.

In all these power projects subsidized by the PWA as well as in the case of larger projects which have been undertaken by the Administration directly, private power companies already in the field have been seriously threatened, and thousands of stockholders who through their faith in fair treatment by Federal and State utility commissions have invested their limited savings in such enterprises, are threatened with serious loss through Federal competition, often in a market already adequately supplied with electrical facilities.

To start the work of the PWA in an intelligent and effective manner required time and, temporarily at least, it did not respond to the immediate need for work relief. To meet this need the Civil Works Administration (CWA) was organized. Under this Administration the municipalities paid in part for materials used, and the remainder of the cost and labor costs were met by the Federal subsidies. This work was not let by contract but was done by day work under Federal supervision. Thus began the diversion of work from its legitimate channels of the construction industry, and the spending of Federal funds for more or less unnecessary and questionable projects.

The PWA was somewhat free from political and local influence, and most projects undertaken were valuable and worth while. The men employed were largely, if not entirely, from the relief rolls, but they were required to perform a proper amount of work for the time spent and the pay received. Under the CWA the work frequently was performed under inexperienced supervision, and was done, in general, at a cost greatly in excess of what could have been paid under the corresponding industry. Even where other work was available, the men generally refused to accept private service which frequently involved less pay for more work as well as the loss of their positions on the relief rolls. Then, too, the selection of projects was far more susceptible to political and local influence, and the insistence of States and municipalities that they should receive their share of Federal funds, often regardless of the necessity or desirability of the work to be done. In this way, Federal funds could be spread more uniformly over the country, meeting more readily the necessity of local employment but without much regard to the real necessity or desirability of the work to be done.

The CWA was succeeded by the Federal Emergency Relief Administration (FERA) and, later, by the Works Progress Administration (WPA) by means of which relief was widespread and many questionable projects were undertaken, often at an enormously large expense. In this manner, billions of dollars were diverted from the construction industry on the revival of which full recovery and the permanent reduction of unemployment so largely depend.

The first relief appropriation of 1933 was for $3 300 000 000. It was commonly expected that all this money would be spent on a great public works program that would revive the construction industry and furnish needed

public works as well as needed employment; but about 25% of this sum or approximately \$825 000 000 was used for CWA and the other day-labor agencies. Of the still greater appropriation of 1934 of \$4 880 000 000, about \$2 500 000 000 was spent by day-labor agencies; and of the 1936 appropriation, about \$2 200 000 000 went for day labor. In the last appropriation of \$1 500 000 000, it is understood this will largely be spent in the same manner. Thus, the Government apparently has spent and will spend for day-labor enterprises about \$7 000 000 000 to the great detriment of the construction industry which under normal conditions and when running at normal rates would employ, in addition to the number now engaged in construction work, an additional force fully equal to the number now on the relief rolls. To the writer it seems doubtful whether the Government has accomplished, by its day's-work program, 20% of the volume of the really valuable work which could have been accomplished had it been done under the PWA and by the construction industry.

Although every right-thinking man must recognize the desirability and the necessity of aiding the needy and taking care of the afflicted, few will agree with Harry L. Hopkins, Administrator, Works Progress Administration, that it is desirable to give the recipients of Government charity a "dignified status" and thus augment the feeling now all too prevalent among those who have received Government aid that the Government owes them a living and that it is a rank imposition that they be required to work at all for what they receive.

Although much has been said concerning intelligent planning and careful consideration of the ultimate results of engineering developments, it is evident that the public works program under construction, so far as its economic aspects are concerned, is the result, not of sound study and thorough investigation, but either of political expedience or irrational inspiration. It seems to be assumed that the ordinary intellect cannot hope to understand the objectives but must be impressed and satisfied by the fact that these projects are the grandest and greatest that the world has ever seen. The fact that some of them are the greatest mistakes that any Government has ever undertaken since the building of the Pyramids, and about as useful, may dawn slowly on the public mind, unless public opinion can be enlightened and freed from false propaganda and political misinformation.

During the decade in which the United States entered the World War many unfortunate enterprises were undertaken. The building of hundreds of ships that were never used; the construction of the Wilson Dam, of an auxiliary steam plant, and of nitrate plants that never were able to function; and the taking over and mismanagement of the railways—all are examples of Government failures which cost the country unnecessary billions of dollars and which should have served as a warning to the people of the necessity of greater intelligence in Congress and of limitations in the authority of the Executive so that vast public expenditures should be based only upon thorough investigation, careful consideration, and intelligent action.

It must be recognized, of course, that emergencies of war and depressions entail conditions which warp the judgment and which confuse intelligent thinking; that under such conditions the people of the nation seem always ready to follow a leader who has positive ideas and who can express those ideas in a

manner that seems to make them rational. The idea seems to prevail that not knowing what to do, people should do something, regardless of consequences. The mistakes of the past have left a heritage of evil. The Shipping Board long added an annual deficit to the burden of the taxpayer, and the Wilson Dam and Power Plant furnished the basic excuse for the establishment of the Tennessee Valley Authority.

Government Interference as a Permanent Policy

Power.—The Tennessee Valley Authority is an undertaking of the Federal Government of an idealistic "unified" development of the valley of the Tennessee River for navigation, flood relief, power development, and various altruistic ideals. Power development as an objective is not a legal function of the Federal Government. It can be undertaken only if incidental to navigation. Power development is undoubtedly the primary objective of the TVA. It is also to constitute a "yardstick" for measuring the cost of generating power for comparison with privately owned power utilities.

The TVA is an interesting project which would not be warranted at Federal expense during prosperous times on account of its purely local character and benefits. Under conditions of the depression, and as an aid to national recovery, it is utterly ill-advised and unwarranted, except as a political gesture of undertaking something big and novel under the excuse of aiding recovery. Like most of the Federal works now in progress, exact and detailed information concerning this project is impossible to obtain, and its detailed ultimate objectives are probably as indefinite in the minds of the Administration as they are to the taxpayers.

As a "yardstick" for measuring the cost of power, this and other Federal hydro-electric power projects are complicated by the fact that "navigation," "flood protection," and other "objectives" are also involved, and that both first cost and cost of development, operation, and maintenance must be assigned to these different objectives. Can it be hoped that the TVA or other Federal Authorities, the leaders of which are responsible for the success of these power projects, will be able to assign fair and equitable parts of the great investments to each of the so-called objectives of the projects, or will power unfairly be found to be a mere by-product of navigation and flood protection?

Arthur E. Morgan, M. Am. Soc. C. E., Chairman of the Tennessee Valley Authority, stated:[10]

> "In case public power is used as a 'yardstick' or as a measure of what the private power industry should charge for its services, then it is imperative that records and accounts be honest and fair and open and that there be no hidden element of subsidy. The very fundamental element of such comparison is honesty, fairness, and openness in measurement. Take away those characteristics and the supposed comparison may becloud the issue rather than clarify it."

All fair-minded engineers will certainly agree with the foregoing specifications for a Federal "yardstick"; but it is fair to ask how it is to be established in practise.

[10] "Public Ownership of Power," by Arthur E. Morgan, *Atlantic Monthly*, September, 1937, p. 342.

On account of the absence of exact data, the TVA project is very difficult to analyze. The best information the writer has been able to obtain is the testimony before the House Appropriations Committee.[11] At this hearing the inquiries of the members of the Committee brought out certain estimates which, so far as the writer is aware, are otherwise unavailable.

The pertinent data given in this evidence are as follows:

Continuous capacity of integrated system, in kilowatts		666 000
Average annual power output, in kilowatt-hours		5 780 000 000
Estimated annual income		$23 120 000
Less annual cost of power	$12 200 000	
Less other expenses	1 170 000	13 370 000
Net revenue		$ 9 750 000
Estimated cost of dams, locks, and reservoirs		$343 700 000
Cost of power-house and equipment		135 450 000
Total estimate for navigation, floods, and power		$479 150 000

In these estimates no allowance is made for interest during construction or for carrying charges while the market is being developed. An estimate of $20 000 000 for contingencies is also omitted as possibly unnecessary (a deficiency appropriation from the Federal Treasury always can be anticipated).

All these expenses are just as much a part of the actual cost of a power plant development, Federal or private, as the cost of buildings or machinery. Taxes during the development period can be ignored in this case, because, in lieu of taxes, Congress has provided that the States shall receive 5% of the gross income—no sales, no taxes; a very generous provision for the TVA.

Based on the estimated income the TVA finds it can pay the entire cost of the project in fifty years, without interest. Interest, however, is as real an expense as capital cost, cost of operation, or cost of maintenance. The estimated net return (without interest), even if it could ever be realized, will not pay interest on any reasonable basis even on the fair cost of the power project alone.

Even if the TVA estimates are correct, the real net income will be negative and the actual cost of this project to the taxpayer will probably be two or more times the total estimated cost before the Federal bonds, issued to meet the cost of this project, are paid.

It is assumed for this analysis that the estimates of continuous power are correct. Such estimates are of necessity always doubtful on account of great variations in annual stream flow, and in this case, are still more doubtful, on account of necessary regulations for navigation and flood control. It should

[11] First Deficiency Appropriations Bill, March, 1936.

also be noted that the annual output is based on the daily use of the continuous capacity for 365 days per yr.

It has been assumed in one critical article written on this subject,[12] that, on account of navigation and the necessary load factor (which can only approximate 50%), not more than one-half this power will be available. The writer believes, however, that with the proposed installation of machinery contemplated it may be possible to generate the full daily output of kilowatt-hours within 24 hr at any reasonable load factor without serious effect on navigation. The effect of systematic efforts for flood control will probably be more serious. The income estimate is calculated on the basis of 4 mills per kw-hr on 100% of the estimated continuous capacity.

From the same source of information the wholesale rates established by the TVA on October 22, 1934, are:

Demand charge, in dollars per kilowatt	0.90
First 100 000 kw-hr, in mills	4
Next 200 000 kw-hr, in mills	3
Next 700 000 kw-hr, in mills	2.5
All more than 1 000 000 kw-hr, in mills	2.0

From these rates it is evident that the annual returns, if and when markets are found for the entire output, cannot average as much as 3 mills per kw-hr and that the returns, therefore, would be more than $5 780 000 less than estimated by the Authority. On this basis the project would require about 125 yr, without interest, to pay for itself. (Press dispatches of August 20, 1937, state that a chemical company has contracted with TVA for 12 000 kw of "firm power" at 2.74 mills per kw-hr and 12 000 kw of "run-of-river power," at 2.27 mills per kw-hr.)

The estimated output is based on continuous efficiencies of more than 80%, which is scarcely possible to obtain under all conditions of head and load. There are other losses due to plant use, transmission losses, transformer losses, and losses due to regulation for navigation and flood prevention. Basing his judgment on a somewhat extended experience the writer believes that if, with a complete market, the Authority realizes 80% of this estimate, it will be accomplishing more than should normally be expected.

The writer recognizes that there is available, in addition to the firm power estimate, a large block of irregular seasonal power that will be available during periods of more than average flow, which the TVA calls "run-of-river power," and which is more commonly termed "dump power." He is also aware that contracts have already been made to supply a considerable quantity of such power. This power can be made firm where auxiliary steam power is installed. No such auxiliary power is proposed by the TVA at present. Whether or not this power project can be made to pay on a fictitious cost estimate is of little moment. The real factor of importance is that the TVA seems to be deliberately attempting to "becloud the issue" and to set up a "yardstick" that is clearly dishonest instead of "honest, fair, and open."

[12] "Black and White Coal," by C. W. Kellogg, *Bulletin*, Edison Electric Inst., May, 1937, p. 163.

In an attempt to show honesty of purpose the cost of power is estimated on the following basis which on its face would seem to be fair: Interest, 3.5%; depreciation, 3%; taxes, 1%; operation, 1.5%; the total being 9% of the cost of power development. The estimate of power house and equipment ($135 450 000 at 9%) equals $12 191 500, which is essentially the same as the annual cost of power previously quoted. This shows conclusively that the cost of the power "yardstick" is estimated at $135 450 000, and does not include any part of the cost of dams and reservoirs. Under such conditions, are these estimates of cost, income, and profits, free from "hidden elements of subsidy"?

The system of high dams is believed to have been built almost exclusively on account of power development, and any honest estimate of the cost of such development should include a large portion of this cost.

The Army Engineers have estimated[13] the average seasonal flood loss in the Tennessee Valley and its tributaries ($1 784 061). What reasonable sum should be expended to prevent this annual flood loss? A fund of about $51 000 000, if set aside at 3.5%, would pay such annual flood damage. The works proposed, however, will not furnish 100% flood protection. Chattanooga, Tenn.,[14] will have to expend $15 000 000 to secure effective relief; and probably other localities may require extensive local works. In any event, an expenditure of $36 000 000 would seem to be about the limit for this purpose, although indirect damages might warrant some additional expense.

The Army Engineers have estimated[15] the cost of navigation works with low locks and dams at $74 709 000. Although the writer doubts the economic desirability of this investment, he agrees that the higher dams will possibly improve navigation facilities and that an increase to, say, $90 000 000 for this purpose might be warranted.

The estimate of cost for the eleven dams and power plants (as of March, 1936) was $479 150 000; the difference in cost between this sum and a fair estimate for navigation and flood protection will indicate the real cost of power development:

Total estimated cost		$479 150 000
Cost of navigation	$ 90 000 000	
Cost for flood protection	36 000 000	
	$126 000 000	126 000 000
Total cost of power installations		$353 150 000

This is more than 2.5 times the estimate of the Authority, and to it should be added interest during construction, carrying charges during development of the power market, taxes (which would be paid by a private company), and presumably cost of transmission, etc.

As a "yardstick" the writer believes that "the supposed comparison may becloud the issue rather than clarify it." It is quite evident that power, the

[13] H. R. Doc. No. 328, 71st Cong., 2d Session, p. 730 *et seq.*

[14] "Flood Control at Chattanooga, Tenn.," by E. L. Chandler, *Civil Engineering*, June, 1937, p. 405 *et seq.*

[15] H. R. Doc. No. 328, 71st Cong., 2d Session, p. 5.

real primary object of these Government ventures, is to be regarded financially by the Administration as a by-product of "navigation" and "flood protection."

The Chairman of the TVA has expressed the opinion[16] that the project is worth all it will cost for navigation and flood protection alone; also that the value of the flood-protection feature will be worth $100 000 000 to flood protection on the Lower Mississippi River. As to the first opinion, it is so extreme that it seems useless to discuss it. As to flood protection value, the writer understands that one primary objective of the TVA is flood protection in the Tennessee Valley. This may or may not co-ordinate with the interest of flood protection on the Mississippi River. The Miami District of Ohio, with automatic reservoir discharge, adds to the floods in the Ohio River about as often as it reduces such floods, depending on the direction of movement, and the length and intensity of storms. Can the TVA accomplish better results?

On the unwarranted basis of the TVA estimates, it is apparent that it estimates the cost of power at about 2.11 mills per kw-hr. On a more reasonable basis of cost of the plants and of the power that can be sold, the probable cost will be from 5 to 7 mills per kw-hr.

Steam power can certainly be generated in the Tennessee Valley for not to exceed 4 mills per kw-hr. As about 65% of the cost of coal is due to labor, it would seem that if recovery was the true purpose of these power developments it would be of greater advantage to use steam power at a much less capital cost as well as a greatly reduced operating expense.

That the power to be generated by this development will never warrant the great expense involved, is further indicated by the findings of the National Resources Committee, which, in its report[17] to the President on June 18, 1937, states that "the high efficiency and the low fixed charges now possible in large fuel burning plants, places hydro-electric developments at a disadvantage in most sections of the United States if low power cost is the objective."[18] (By a majority vote the Tennessee Power Commission has granted permission for the construction of a $2 000 000 steam generating plant in Nashville.)

If there is any honest and intelligent defense for the expenditure of approximately half a billion dollars which is apparently now being largely wasted in the Tennessee Valley, it should certainly be made manifest. This is especially true if the Administration ever expects to secure the backing of public opinion for repeating this seemingly irrational experiment in six other sections of the country.

It seems apparent that the TVA is only slightly less absurd than the Passamaquoddy Project which, however, it greatly exceeds in cost. It is quite equal to the Grand Coulee Project by which it is proposed to develop not only an enormous amount of power for which there can be no market for years to come, but also to irrigate more than a million acres of land, largely in private ownership, for which there will be no demand within a still longer period.

Reclamation.—Almost the entire record of irrigation under Federal laws and Federal control in the United States has been a series of economic failures.

[16] Testimony before Appropriations Committee, 1936.

[17] *Electrical World*, August 23, 1937, p. 260.

[18] *Loc. cit.*, p. 52.

The Carey Act of 1894 resulted, by 1925, in the irrigation of only 1 158 926 acres under private and corporate efforts. In general, the cost of bringing water to the land was so great that success was practically impossible. This was due (in part, at least) to promoters' profits. Such lands were partly colonized through misrepresentations, and, in many cases, the settlers lost everything in their attempt to cultivate the land.

In 1902, reclamation was started by the Government as a self-sustaining and self-liquidating venture. These projects were to be financed from a revolving fund. The time of payment was first planned at 10 yr, then 20 yr, and, finally, at 40 yr, but the funds refused to revolve.

The following description[19] of the situation that followed can but remind the reader of recent history. Such results are, with few exceptions, the indispensable condition of all Federal ventures into business:

"Once irrigation work got under way, every state west of the 100th meridian besieged Congress to approve one or more pet projects. When government money was to be had for the asking, it put a new zest into politics. Here was something tangible to work for—pie to bring the faithful back home. * * * In consequence every patch of brown sagebrush became a prospective Garden of Allah in the mind of some one. All it needed was a Moses with his staff to make the water gush forth, and lo, the desert would burst into bloom.

"Those were grand days for the promoters, the real estaters, the speculators, the chambers of commerce, the business men and the politicians. Of the latter every one who carried a staff aspired to be a Moses, and he was the most honored and the most loved who was able to bring forth the biggest fountain.

"Every project had a political father and mother and a host of godfathers, sons, uncles, and other close relatives. * * *"

Flood Control.—The space available will not permit further consideration of Federal public works now under construction. The writer desires, however, to consider briefly the matter of Federal flood-control works that apparently is soon to become important. An appropriation of $34 177 000 chiefly for flood control on the Ohio River has already been passed and approved. This is only a start on the vast expenditure that may be involved in this important problem.

The public has many erroneous ideas concerning the cause of floods. When the writer was one of the commission called to China, in 1914, to determine what could be done concerning the control of the Huai River which frequently inundated its lower valley, it was found that the people of China believed that these floods were produced by the Hydra, a mythical dragon, which they believed dug into the hills and let out the flood waters. Even the average citizen of America smiles at such a crude idea but, nevertheless, believes in the no lesser superstition that floods in America are caused by "deforestation." It is apparently not generally known that floods, quite equal to any that have recently happened, occurred long before the forests of the United States were cut for timber. Neither does it seem to be understood that devastating floods occur only in valleys and lowlands which have been built up by the rivers in the past for the purpose of passing their flood waters. When mankind builds cities, factories, or farms on such lands, troubles from

[19] Editorial in the *Country Gentleman*, January, 1927, p. 26.

the occasional high floods must necessarily be expected. Flood control is, in fact, an attempt to save the people from the results of their own folly and ignorance, not in the cutting of timber, but in attempting to usurp the lands which the rivers occasionally need to discharge their maximum floods. In most cases, these lands, convenient to the river for the purposes of navigation, water supply, and pleasure, are the most accessible for railroads, highways, and buildings during normal river flow, and the most attractive sites for cities. The farm lands of the valleys and flood-plains, often greatly enriched by silt deposits, were naturally sought for farms. The great floods usually occur only at long intervals of time. The damages due to occasional floods were not great when the country was young and the people few in number. With the great growth of the centers of population and manufacturing often located in the exposed positions, flood losses have become enormous.

Some communities that have experienced heavy losses in property and life, such as the Miami River Valley, have had sufficient energy and foresight to accomplish flood control at their own expense. Under present conditions of Federal spending, it is only to be expected that such communities are now calling on the Federal Government to save them from their own short-sightedness. The Federal Government has already committed itself to power development and flood control under the guise of navigation. Almost everywhere that floods are experienced, navigation either at the point where the damage occurs, or on the main river below, may be used as an excuse for Federal expenditures.

The writer believes that, in the future, with the precedent of the TVA and other recent Federal projects where flood control has been named as one of the objectives, the Federal expenditures for flood control will be one of the great items of Government expense. It is to be hoped that both public and political opinions may be so informed that their great and important works will be handled in an intelligent and economical manner.

In the last few years so much has been heard of "little waters," "up stream engineering," and "holding each drop of water when it falls," that at times it has looked as if flood control would be attempted by means of a few small reservoirs at the head-waters of streams, by a few small dams to prevent erosion, and by the planting of forests.

In his message to Congress, on floods, President Roosevelt has stated, however, that floods cannot be prevented by forests alone or by dams alone, but that such control demands the consideration of many factors. He has deferred action on the plans of the Army Engineers for certain flood-control projects until other interested Departments of the Government could be consulted. It is quite certain that the theoretical values of forest, improved farming methods, and erosion control will not be overlooked when these projects are finally considered.

The writer fully appreciates the importance of forestry, erosion control, and improved farming methods. All are important and should receive due consideration in their time and place. There is no question but that the creation of forests on the hills and mountains, the grassing of sloping fields, and the construction of check dams for preventing erosion, have an effect on

run-off during moderate rains and will distinctly reduce ordinary run-off and even light floods. With frozen or saturated soils, however, and the downpour of maximum flood-producing rainfalls, such preventives of flood run-off are only as effective as footprints in a field, which may hold back water to a small extent, but are so utterly insignificant as to be unworthy of any consideration under these extreme conditions.

There are no men more interested in every means of reducing maximum floods than engineers who have studied the subject of flood control. The effect of forests on stream flow has been carefully investigated and thoroughly studied many times, but the writer has yet to be convinced from such studies that deforestation or reforestation has ever had the slightest practical effect on the maximum flood of any stream.

Forests for pleasure and profit, and for the prevention of erosion, have real value; and reforestation for its real intrinsic value has the earnest support of the writer.

The study of cause and effect is a somewhat difficult and intricate matter. The writer was in China at the time of a lunar eclipse. The natives thought that a dragon was attacking the moon. Fire crackers were exploded, gongs were beaten, and pandemonium broke loose. The results were apparently fully satisfactory. In a short time the trouble was over. The moon came forth serenely, even the part that had apparently been bitten out by the dragon was replaced. The moon was saved to the satisfaction of the Chinese people as it has been many times in the 4 000 yr of Chinese history and as it probably will be for hundreds of years to come. Such ideas are not confined to the Chinese. The writer found in the *Proceedings* of the Society not long ago the statement that the author knew that forests produced rainfall because he had frequently stood on the sunlit lowlands of Central America and had seen heavy rainfalls over the forests of the foot-hills and mountain slopes. Did the forests produce this rainfall or do trees normally grow where rain naturally falls? The people of the United States may, perhaps, receive a somewhat expensive answer to this question when they have the final results of the shelter belt, which, the writer understands, is still being planted on the Great Plains in spite of the refusal of Congress to appropriate money for that purpose.

Responsibility of the Engineering Profession.—The welfare of the future depends much on the Engineering Profession. The engineer, through his training and through his experience in connection with past enterprises, is best fitted to make a truthful analysis of the results which must be anticipated from most of these great projects. If he will assume the responsibility of informing the public concerning those matters, he can assist materially in creating and arousing an intelligent public opinion that will demand sane and intelligent consideration of future governmental activities.

The members of various Engineering Societies in Chicago, Ill., recently voted to subject Government public works projects to critical examination by competent members and to publish the findings. Similar action should be taken by every Engineering Society in America. Much good can be accomplished by such action. Will the Engineers and Engineering Societies of America rise to this important duty?

The engineering features of the TVA, of navigation, and other works done under the Army Engineers, of the Reclamation projects, and of such of the present projects, as the writer has seen, are, in general, well designed and well constructed and he has nothing to criticize in this regard. Fortunately to this extent politics has been eliminated; but sound engineering cannot make an uneconomical project an economic success.

Government projects, both in the present and in the past, have been based most largely on political expediency, especially when the Government has ventured into business enterprises. This has been true regardless of the party in power, whether Federalist, Whig, Democrat, Republican, or Progressive. The principle of spreading the spoils of Federal activities over the entire country in order to satisfy the electorate, regardless of any real justification of the individual projects involved, has become too well established and, in the opinion of the writer, is the only real explanation of many Federal activities of both the present and the past.

The Grand Coulee Project, as an immediate objective, was so utterly condemned by official reports of the Army Engineers, the Reclamation Service, and the Department of Agriculture before it was undertaken under executive order, that it seems hopeless to anticipate that Government undertakings can ever be based on sound economic principles, unless an intelligent public opinion can be both created and aroused to active influence in municipal, State, and Federal activities. The conditions prevailing at present are certainly a sad reflection on the present so-called civilization. Unless the thinking men of this country, the men who are willing to stand for a square deal, for sanity, for truthfulness, honesty, and intelligence, aside from party and aside from personal interests, soon begin to take an active part in public affairs, the writer sees little future hope for the success of democracy or of the American nation.

HAZARDS OF UNECONOMICAL CONSTRUCTION

By Henry Earle Riggs,[20] President, Am. Soc. C. E.

Synopsis

The magnitude of the unemployment problem which faced the present Administration when it came into power, the country-wide extent of the relief program, the fact that billions of dollars have been paid out since 1933 either in the form of direct relief or for many hundreds of projects of work relief, and the multitude of different agencies for the distribution of relief funds—all of these influences have tended to divert the attention of the general public from the vast program of major public works which has been entered upon as part of the relief program by various Federal agencies.

Many of these great undertakings are wholly unknown to the public. Where they are located; what they are for; the great magnitude of some of them; the cost of them running into tens or hundreds of millions of dollars; the annual costs of interest, maintenance, and operation that will be incurred after they are completed; the assurance that they have of success or failure—all of these phases of the problem are a closed book to the great majority of taxpayers. The non-technical visitor to the construction work is awed by the immensity of it and is furnished with printed information, some of it in the nature of propaganda, which emphasizes the great size but is silent on all matters of economics.

What kind of projects should be adopted for the purpose of furnishing work relief? Is the nation justified, in a time of great stress, and of rapidly rising national debt, in engaging in business ventures that are new to Government in the United States? Is it economically sound to spend vast sums of money in building works for the far distant future, which are distinctly not needed now? Is it wise to build without long and careful planning when the building involves tens of millions of dollars? Is a project economically sound when it may be seriously questioned whether there is a public need for it, a demand for its services, or a doubt as to whether it can be self-liquidating?

These and other similar questions which are suggested by the present program of public works should be proper matters of discussion by the Engineering Profession. This paper is an attempt to present the available evidence not only as to a few of the projects approved in the four years, 1933 to 1937, but also of some projects of earlier administrations which undoubtedly can be classified as failures of publicly owned State and Federal business undertakings.

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It is written for the purpose of directing attention to the need for long-time planning, by Congress, of Federal public works and of the necessity for establishing definite requirements as to accounting, publicity of statistics, and the elimination of subsidy in connection with the operation of any publicly owned business or public-service enterprise.

No criticism is made or is intended to be implied of the design or construction methods of the projects discussed. The work of the engineers on the design and the handling of the field work of construction on all the major projects which the writer has visited, is excellent. Bonneville Dam and the dams on the Tennessee Valley Project are fine examples of modern engineering skill. Grand Coulee Dam and Fort Peck Dam will be the greatest structures of their kind in the world. They will continue as great monuments in the desert to the ability of American engineers to overcome well-nigh unsurmountable difficulties.

The construction plant that is used on these major projects and the construction methods and administration come in for equal praise. The most modern types of labor-saving equipment are in use. The impression one gains on a visit to the work, whether it is done by contract or by force account under the Corps of Engineers of the U. S. Army or other Government agency, is one of utmost efficiency of men and machines. Whatever there is of future credit on account of these undertakings will belong to the designers and the builders.

The question is whether such projects are economically justifiable as relief undertakings. In many cases, projects have not been selected by the Congress and, in some cases, they have been adopted in the face of adverse recommendations by competent engineers. The fact that the construction of many of the projects constitutes an entry of the Government into direct competition with private enterprise in the electric power or transportation industries has no bearing on any of the criticisms made herein. The writer is a thorough believer in the regulation of public service corporations and in the public ownership of utilities whenever the public authority can give better service, or lower rates, or can safeguard health or other public interests, more effectively than private corporations. He is by no means convinced, however, that Government ownership and operation of great and complex utilities is likely to be as successful or as efficient as fully regulated private ownership. A study of the results of Federal and State ownership does not tend to increase confidence in the success of many of the businesses thus involved.

This paper is an attempt to present exact facts regarding certain selected projects, both old and new; to present analyses and comparisons which throw

light on the economic phases of the projects; and to indicate the financial hazards attendant on the hasty adoption of projects for great public works for which no complete surveys have been made, for which final plans and estimates are not available, and for which not only have no contracts for the sale of the product been made but, often, no present market for the product is in sight.

Passamaquoddy Tidal Power Project

The first example of present-day projects selected is the Passamaquoddy Tidal Power scheme, situated within two miles of the Canadian International Boundary on the Maine coast. This is an electric power project. As originally adopted by the Public Works Administration (PWA), it was a modification of a promotor's scheme which had been under discussion for about twenty years. The plan was to develop power from the high tides at the mouth of the Bay of Fundy by creating a reservoir out of Cobscook Bay, in Maine, which should always be held near high-tide level. Power would be generated by releasing water from this reservoir through turbines after the tide had fallen 4.5 ft, and continuing through low tide and until the rising water had returned to a level of 4.5 ft below that in the reservoir. This would permit power development for about 7 hr out of every 12. Because of the constant change of head during the 7-hr period and the fact that in each month the range of tides varies from about 10.5 ft to 24 ft, there can be no possible uniformity of power output, and, consequently, a small amount of firm power when the total installed capacity is considered.

To supply power during the 5 hr of each 12 when tidal power is not available it was proposed to build an artificial reservoir about 120 ft above sea level, to use the secondary power developed during the 7 hr of tidal operation to pump sea water into this high-level reservoir, and to permit it to run back into the ocean and operate turbines during the 5-hr period and thus furnish the power required for the stand-by plant.

The original plan for an international project was rejected in 1925 and 1926 by responsible private interests. The modified plan was submitted to the PWA in 1933 and twice rejected, and again turned down as unjustifiable after hearings by the Federal Power Commission. The original request to the PWA was for \$47 000 000.

On May 16, 1935, Harold Ickes, Secretary of the Interior, announced that his Allotment Board had approved an allotment of \$10 000 000 initially on an estimated total cost of \$36 284 000.

The task of building the project was assigned to the Corps of Engineers, U. S. Army. Preliminary construction work was begun at once, and at the same time extensive surveys, explorations, and research studies were carried on and an estimate of cost was made. That estimate, which has never been made public, is known to approximate \$62 000 000. Shortly after it was submitted, a commission of three nationally known and thoroughly competent hydraulic engineers (Charles H. Paul and Joseph Jacobs, Members, Am. Soc. C. E., with

Professor William F. Durand, of Stanford University) was appointed and directed to review and reconcile all estimates of cost. That report likewise has been filed and not made public. It approved in general the plans proposed by the Corps of Engineers, and the estimate of the cost of construction was somewhat higher than was estimated by the Army Engineers.

It does not seem out of place to emphasize here the fact that these are but two of a large number of engineering reports which should be available to members of the Congress, the public, and the Engineering Profession but which repose in the files at Washington, D. C.

The estimate of $62 000 000 was based on the original plan which included the high-level pumped storage reservoir and was for the same work covered by the estimate of $36 284 000 given out by the Secretary of the Interior at the time the project was approved. The Army Engineers proposed a revised plan which eliminated this high-level reservoir and substituted therefor a Diesel or steam plant as an auxiliary source of power to be used when the tidal power plant was non-operative. The estimated cost of this revised plan was approximately $37 000 000.

A paper by Captain Hugh J. Casey, Corps of Engineers, U. S. Army, M. Am. Soc. C. E., presented before the Boston (Mass.) Section of the American Institute of Electrical Engineers,[21] stated that the tidal plant will be capable of producing "a continuous output of 30 000 kw on a 100% load factor" and that "the combined plants will be capable of an annual net output of 260 000 000 kw-hr of prime energy."

It is obvious that either $62 000 000 or $37 000 000 for a plant with a firm power capacity of only 30 000 kw is many times the cost of a modern steam plant. It is equally obvious that if a steam or Diesel plant is built for stand-by use during 5 hr out of every 12 there is no reason why it should not operate all the time. If this stand-by plant is built it reduces the tidal power part of the project to an absurdity. It is evident that none of the engineering boards that studied this project for private interests or for the PWA between 1924 and 1933 could find any market that would warrant such an expenditure. The data published by the Federal Power Commission indicate beyond any question that there is no present market for this power in Maine.

In July, 1935, the Hon. Louis J. Brann, Governor of Maine, appointed a commission composed of Kenneth C. M. Sills, President of Bowdoin College; and Messrs. W. K. Campbell, W. F. Cram, H. B. Crawford, and E. S. French. The report[22] of the Commission which finally secured favorable action by the Administration made no attempt to justify the project as economically sound This report stated:

"We would argue for federal support for this project largely on the grounds of social desirability, that is, of benefit in the long run and in many ways without definite assurance that the project would be within a definite number of years self-liquidating. * * * In the present economic situation private capital would not be available and the government has already denied a loan to a private corporation on the ground that a sufficiently sure market for the power generated has not been shown."

[21] Not published.

[22] *Portland (Me.) News.*

Approximately $12 000 000 was allocated to this project. Work on housing, surveys and explorations, research studies, and some minor construction work—all costing slightly more than $6 000 000—was done before Congress refused to make any further appropriations.

Improvements for Navigation

The second illustration that comes to mind is a navigation project. Navigation on the Missouri River by stern-wheel steamers commenced prior to the Civil War. It was of major importance to the West in the days before the railroad and, although the tonnage moved was not very large, it continued during the period 1860 to 1885, when the railroad network had fully covered the district between the Mississippi River and the Rocky Mountains.

The first appropriations for improvement work on the Missouri River were made under the River and Harbor Act of August 14, 1876,[23] and the original plan for comprehensive improvement of the river was adopted August 2, 1882.[24] The earlier Republican Administration initiated these expenditures on the Missouri River; and the present projects were adopted or modified in 1912, 1917, and 1927. Only the Fort Peck Dam Project was adopted under the present Administration. Therefore, prior Administrations must share any criticism as to the propriety of the great expenditures on this river. Over the years from 1876 to June 30, 1933, the total expenditures for new work and maintenance was a little less than $95 000 000, and during those years it was termed by the critics, "pork barrel legislation."

On October 3, 1933, "it was recommended to Congress that the project for navigation on the main stem of the Missouri River as heretofore authorized, namely from the mouth to Sioux City, be vigorously pressed to completion, and that, in addition the reservoir at the site of Fort Peck be built to the maximum practical capacity, and be operated primarily for navigation."[25]

During the first three years of the present Administration $47 000 000 was spent on this river improvement, and more than $60 000 000 of an estimated $109 000 000 was spent on the construction of Fort Peck Dam,[26] so that the net total of Federal money spent for navigation on the Missouri River to June 30, 1936, was $202 495 633, and the total appropriations amounted to $248 232 000, leaving $45 736 000 of available funds for work in the fiscal year 1937. To complete Fort Peck Dam another $18 500 000 will have to be appropriated, bringing the total for navigation on the Missouri River to $267 000 000, or more.

At least 90% of the reported traffic on the Missouri River was material for Government construction, and has been for the past eight or ten years. In 1929, when total traffic was reported as 1 372 000 tons, all except 6 155 tons was noted as "used in river improvement and moved by owners." Since that year specific items used in Government construction are not indicated but the general footnote appears in each yearly report, as follows, "a large percentage of the traffic reported was for river improvement work."

[23] Rept. of Chf. of Engrs., U. S. Army, 1933, Pt. I, p. 712.
[24] *Loc. cit.*, p. 707.
[25] *Loc. cit.*, p. 708.
[26] Rept. of Chf. of Engrs., U. S. Army, 1936.

There can be no possible criticism of reasonable Federal expenditures for keeping navigable and navigated waterways and harbors in condition for use. Comparatively few such waterways can be self-liquidating and the value of any of them to the public is incapable of exact determination, such value being principally in the form of modified rail rates over large areas due to the competition of water transportation. One yardstick of the economic justification of these expenditures on the Missouri River can be had by comparison with like expenditures on the Great Lakes, and by a comparison of the tonnage and value of freight moved on the two waterways.

The total cost of all capital and maintenance expenditures on the Great Lakes between 1824 and 1936, including all work on the harbors of the Great Lakes and the navigable rivers entering them, all connecting rivers such as the Detroit, St. Clair, and the St. Marys, with the locks at the "Soo," in short, all Government expense of every kind on this great system of internal waterways, was \$248 277 000, or almost exactly the same as the appropriations on the Missouri River to the same date (\$248 232 000).

In 1935 gross total traffic at all Great Lakes ports totaled 186 879 000 tons, or more than 101 times as much as the 1 841 000 tons reported on the Missouri River, nearly all of which was construction material, and therefore, temporary traffic. The total value of Great Lakes freight was \$2 913 070 000 which is 236 times as much as the \$11 941 000 total freight value on the Missouri River. If the expenditure of \$95 000 000 for navigation on the Missouri in a period of 57 yr was "pork barrel" is not the appropriation of \$153 000 000 in three years "pork barrel supreme"?

Fort Peck Dam

Fort Peck Dam has been considered as part of the Missouri River navigation project. Further brief reference to it appears to be necessary. This great structure is about 1 600 miles northwest of Kansas City, Mo., in North Central Montana. It will be the world's largest earth dam, requiring more than 100 000 000 cu yd of earth and gravel in its construction, with spillway, tunnels, and auxiliary structures of equally imposing magnitude. It began with an estimated cost of \$86 000 000.[27] The revised estimate of 1936 places the cost at \$108 600 000.[26] It is expected that all major structures will be finished by June 30, 1938, and that the dam itself will be finished and placed in operation in the autumn of 1939. It is a magnificent piece of engineering design and construction, and no possible criticism can attach to the builders of it.

Although the plans provide for a possible future power plant there is small likelihood of one being built except for irrigation pumping as the territory is sparsely settled and there are not more than three or four towns with more than 5 000 population within a 150-mile radius. Ample power is now developed in the western part of the State to meet every requirement for years to come. No present irrigation development is proposed, although, like power, this is a possible future by-product of the dam.

[27] Rept. of Chf. of Engrs., U. S. Army, 1934.

The Division Engineer of the Corps of Engineers, U. S. Army, is quoted[28] as placing $3 000 000 as the justifiable expenditure at Fort Peck for flood protection in the river below St. Joseph, Mo., and he states, further, that "it would have little if any effect upon Mississippi floods."

This clearly is a navigation project. It will insure the maintenance of a minimum flow of 30 000 cu ft per sec at Yankton, S. Dak., and at points down stream. Power, irrigation, and flood control are practically "out of the picture."

This dam will be completed within a year or two. Water is now (1938) flowing through the tunnels, all the auxiliary structures are nearing final completion, and there will be left after this year only the completion of the earth dam.

Power in the Northwest

In the summer of 1933, in the first wild rush to find projects for putting the unemployed to work at once, $63 000 000 was allocated by the PWA for the construction of a low dam, which later could be increased in height, across the Columbia River at the head of the Grand Coulee, in Northern Washington, and the building of a power "yardstick" for the Northwestern States. This project had been estimated by the Army Engineers to cost about $108 000 000.[29]

Politics "got to work" as soon as this project was approved. The State of Oregon had a power and navigation project for which it demanded approval, and within a few weeks $31 000 000 was allocated by the PWA to the Bonneville Dam and Power Plant, which was close to the power market, which alone was capable of furnishing as much power as will be needed in the area, and which extends deep-water navigation about fifty miles to The Dalles, Ore.

Construction at Grand Coulee was put in charge of the Bureau of Reclamation, a contract for the dam and power plant was entered into on July 16, 1934, and work on the dam was actively begun.

The fact that the Government had commenced work on two power projects, either of which was capable of meeting all reasonable present or anticipated needs for additional power in Washington, Oregon, and Idaho, the three States having the cheapest power in the country and the greatest per capita consumption of power, indicated the necessity for a reconsideration of the original plans for Grand Coulee Dam. Therefore, extended studies were made by Engineers of the U. S. Bureau of Reclamation, and on June 5, 1935, an official order was issued changing the contract from one for the low dam and power house to a contract for the foundations of an ultimate high dam[30] which would back water to the Canadian Boundary and constitute the first step in a gigantic project to irrigate more than a million acres of land known as the Columbia Basin.

The plan adopted by the Bureau was one which promotors had been urging for a number of years, in which irrigation was the final objective, but irrigation which could only be justified if it was subsidized by the sale of vast quantities of power. The order made most revolutionary changes in the contract,

[28] *Engineering News-Record*, November 29, 1934.

[29] House Doc. No. 103, 73d Cong., 1st Session, p. 735.

[30] Paper by Kenneth B. Keener, Senior Engr., U. S. Bureau of Reclamation, *Engineering News-Record*, August 1, 1935.

substituting foundation work on a much larger dam, for the originally planned complete low dam and power plant, all in the bed of a swift river with a mean flow of more than 100 000 cu ft per sec, and a maximum recorded flow of 460 000 cu ft per sec,[31] involving increases in quantities of more than 3 000 000 cu yd of common excavation and more than 850 000 cu yd of mass concrete. The contractor was to do this work for the same price as that named in the contract for the original project "subject to adjustment."[30]

The Columbia Basin Reclamation Project, substantially as now contemplated by the Bureau of Reclamation, was estimated by the Corps of Engineers, U. S. Army, in its study of power, irrigation, and flood-control possibilities on the Columbia River, as follows:[32]

Total for power and navigation	$215 483 453
Total for irrigation	221 721 880
Total power, navigation and irrigation	437 205 333

This was a preliminary estimate and, in view of the unprecedented difficulties of construction, may conceivably be substantially exceeded. A statistical abstract of the "United States Under Government Irrigation Projects" shows the charge to irrigation of $11 419 070 of the cost of Grand Coulee Dam to June 30, 1935. Inasmuch as the entire cost of the river dam is charged to "Power" in this estimate the final distribution of cost will be very different from these data.

The present work under contract includes only the foundations of the Columbia River Dam, of which Kenneth B. Keener, Senior Engineer of the Bureau of Reclamation, states:[30]

"* * * completion of the Grand Coulee Dam is a problem for the future and one that need not be given serious consideration until the work now under contract is nearing completion. Conditions and requirements at that time will no doubt govern the next step to be taken in the development of the Columbia Basin project."

Within a year or two at the most Congress will be asked for more money to continue this project. That body may very properly ask just what it is proposed to do, why it is needed, and whether the project will be self-liquidating. The dam is advertised in literature given out to visitors to be "the world's largest masonry structure, larger than the great pyramid, four times as long as Boulder Dam." It will be 500 ft high, 4 300 ft long; it will contain 11 250 000 cu yd of concrete, and will involve 16 000 000 cu yd of excavation. According to a tabulation of the Bureau of Reclamation the power plant will have a rated capacity of 2 500 000 hp, and will be able to develop 7 000 000 000 kw-hr of firm power and more than 4 000 000 000 kw-hr of secondary power per yr. This rated horse-power capacity is almost 16% of the rated capacity of all the water-wheels in the United States,[33] and more than the 2 358 000 hp of the California turbines. It is made perfectly clear in the report of the U. S. Corps of Engineers that "the feasibility of the pumping plan [this project] is de-

[31] Rept. of Div. Engr., H. R. Doc. No. 103, 73d Cong., 1st Session, p. 58.

[32] H. R. Doc. No. 103, 73d Cong., 2d Session, p. 15.

[33] Given as 16 179 475 hp as of January 1, 1937, in the Dept. of the Interior Press Memorandum, dated February 16, 1936.

pendent on the possibility of selling practically the entire output of primary power from this plant at a remunerative rate."[34] The late Elwood Mead, M. Am. Soc. C. E., has stated:[35] "This development to be solvent must be based on revenues from power and these revenues must contribute to the cost of the irrigation works to avoid injurious burdens on irrigation farmers." Therefore, the first consideration as to this plant is the question of market for power.

In considering the market it must be remembered that Bonneville, in Northern Oregon, is in the same territory. The Bureau of Reclamation states that 7 000 000 000 kw-hr of firm power will be generated at Grand Coulee. The statement has been made[36] that the two dams on the Columbia—Bonneville and Grand Coulee—will have a combined final installed capacity of 2 322 000 kw and can produce 10 220 000 000 kw-hr.

The combined capacity of the two plants is greater than that of all classes of generating equipment of any State in the Union in 1935, except only New York and California. It is 840 000 kw in excess of the total equipment in Oregon, Washington, and Idaho, in 1935.[37]

The total power generated in that year in the eight States of Oregon, Washington, Idaho, Montana, Wyoming, Colorado, Utah, and Nevada was 6 772 000 000 kw-hr or less than the firm power capacity of Grand Coulee alone. The total power generated in Oregon, Washington, and Idaho, which is where these plants must find their market, was only 4 570 281 000 kw-hr.[37]

Need anything more be said? Any study of the power market in the Northwest indicates that there would be difficulty in finding a market at present for 1 000 000 000 kw-hr, and the market for any such amount as would justify Grand Coulee as a power project is more than a generation in the future.

The Columbia Basin Project

This Grand Coulee Project cannot be dismissed without emphasizing the fact that, in 1927, six years before the emergency approval of Fort Peck, Grand Coulee, and Bonneville, the Congress attacked the problem of the wise and proper development of the Columbia River in a highly proper manner. In that year an Act was passed providing for an exhaustive study of certain rivers, including the Missouri and Columbia, with the view to determining the possibilities for the development of navigation, flood control, power, and irrigation, on these great rivers.[38] One of the so-called "308 reports" dealing with the Columbia River is dated July 31, 1931, and was available to the Federal officials in 1933 when Grand Coulee was approved as a project.[39] This report discussed the building of either eight or ten dams which would develop more than 92% of the total head of the Columbia between the International Boundary and Tidewater. Of these dams, one at Warrendale, Ore. (or Bonneville as an alternate site), was the lowest, at Tide-water, and the one at Grand Coulee was the last one before reaching the International Boundary. The Corps of

[34] H. R. Doc. No. 103, 73d Cong. 1st Session, p. 8.
[35] *Loc. cit.* p. 5.
[36] *Electrical West*, February, 1935.
[37] *Statistical Bulletin No. 4*, Edison Electric Inst., January, 1937.
[38] H. R. Doc. No. 308, 69th Cong., 1st Session.
[39] H. R. Doc. No. 103, 73d Cong., 1st Session.

Engineers of the Army went at length in this report into the consideration of building Grand Coulee in connection with the Columbia Basin irrigation. Apparently, the fact that this was a promoter's project was the reason for the exhaustive study of it.

A careful analysis of this report disclosed strong adverse recommendations to the project now under construction; for example, Maj. Gen. Lytle Brown, U. S. Army, M. Am. Soc. C. E., then Chief of Engineers, U. S. Army, stated:[35] "In fact the local reports demonstrate that the irrigation of land as pertains to the Columbia River area under consideration is not an economical proposition at this time and should await the future."

The Board of Engineers for Rivers and Harbors stated:[34]

> "One large area of upland known as the Columbia Basin could be irrigated by gravity from the Clark Fork and Spokane River or by pumping from the Columbia River by means of secondary power developed at the plant near Grand Coulee. The feasibility of the pumping plan is dependent on selling practically the entire output of primary power from this plant at a remunerative rate."

Subsequently, the Board discusses this subject at length, as follows:[40]

> "The economic feasibility of this plan is largely based on subsidizing irrigation by profits from the sale of power. * * * The Board * * * is of the opinion that the estimates of the reporting officers that the growth of power demand will be such that the entire prime output would be absorbed in a period of 15 years after 1940 * * * is unduly optimistic. * * * The Board * * * is unable to recommend the adoption of this project at this time."

This report, dated March 29, 1932, was available when the project was decided upon and is full of sound argument why Grand Coulee should not be built. A "Report on the Columbia Basin Project," dated February, 1925, two years prior to the action of Congress ordering the Corps of Engineers, U. S. Army, to make its study, was addressed to the U. S. Bureau of Reclamation by Louis C. Hill, Past-President, Am. Soc. C. E., Joseph Jacobs, Charles H. Locher, Richard R. Lyman, and A. J. Turner, Members, Am. Soc. C. E., and Mr. O. L. Waller. This Board found that economic conditions definitely favored the adoption of a gravity project by diverting water from Clark Fork at Albany Falls, which would cost \$158 per acre to irrigate 1 224 000 acres, and did not deem it necessary to make an elaborate analysis of the Grand Coulee scheme.

Discussing Grand Coulee on March 19, 1912, Elwood Mead stated:[35] "It will require at least ten years after the works are authorized, to build the dam and power plant, and another ten or fifteen years to absorb the power thus made available. These things must precede the large expenditure to build the works required for irrigation."

It is impossible in such a paper as this to brief all the evidence fully. To the writer the essential points appear to be as follows:

(1) In 1925, a study of the irrigation project was made by the Hill Commission which recommended a gravity system at a cost of \$158 per acre. This report has not been made public, but apparently the total cost of the gravity plan approximated \$195 000 000.

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[40] H. R. Doc. No. 103, 73d Cong., 1st Session, p. 12.

(2) In 1927, the Congress ordered the Corps of Engineers of the U. S. Army to make a study of the entire Columbia River and to report on the possibilities of navigation, irrigation, power, and flood control. This was done and an 1 845-page report was submitted which considered this specific project at length, and which did not approve it.

(3) It appears from various sources that the larger part of the land in question is in private ownership, much of it in the hands of bankers and speculators.

(4) The project as first approved by PWA was for an entirely different plan involving only power but with a low dam which, subsequently, could be raised in height. Later, this plan was abandoned and the change to the large power development to support the irrigation scheme was made by the Bureau of Reclamation.

(5) It does not appear that the project was presented to the Congress before its adoption or discussed by Congress in the light of the report to it by the Corps of Engineers, and with full understanding of the magnitude of the undertaking.

(6) The Corps of Engineers' estimate of cost of the present project was $437 000 000. The Statistical Abstract for 1936 shows the total cost to the Government of all the forty-five other reclamation projects of the Federal Government, excluding all charges to the Grand Coulee project, to have been less than $237 500 000. Based on charges to date to irrigation on this project, it is evident that this one irrigation project alone will cost more than all previous Federal reclamation projects, and the entire project will exceed the cost of the Panama Canal by many millions of dollars.

(7) A study of existing power development in Oregon, Washington, and Idaho, a large percentage of which is publicly owned by the Cities of Seattle and Tacoma, Wash., and of the production and sale of power in these States, seems to indicate that there is no possibility of the sale of all the power of both the present Bonneville installation and Grand Coulee; but all the power of Grand Coulee must be sold at a profit to subsidize irrigation in order to justify the scheme.

It is to be hoped that the Congress will not take hasty action in approving further appropriations for this project without the most complete expert study. The request for more money, which will inevitably be made by 1938, should be cause for an exhaustive study and a report which will be available to the public by a commission of nationally known, highly qualified, and wholly disinterested engineers. If there is no market for practically all the energy (and that appears to be the fact) the entire project will fall.

Freely granting the urgent need that existed in 1933 to find public works to relieve unemployment, the writer is of the opinion that such a project as this one, which will require years to complete, and which may involve half a billion dollars, does not fall into the class of emergency relief works. It should never have been adopted without the full discussion and consideration of Congress.

"Counting the Cost"

The nation will do well to count the cost of the great program of public works which has been commenced. The sums of money published as the estimated cost of construction are imposing enough in themselves, and would total somewhere between $1 000 000 000 and $2 000 000 000, if all the major projects were listed (such as the ones already named, and others like the Tennessee Valley Authority, the Florida Canal, Central Valley, California, and other Federal projects, and such Federal financed State undertakings as the Central Nebraska Power Project, the South Carolina Public Service Authority, and the Lower Colorado Authority in Texas).

In "counting the cost" it is not the intention herein to consider anything except actual cost which sooner or later must be paid in the form of taxes. Possible losses of private capital invested in power properties or transportation agencies whose business may be injured and whose credit may be wrecked by competing Government agencies, or the possible destruction of industries such as that of the salmon fisheries by the building of dams, may be substantial, but they are controversial matters which will not be discussed.

Too much emphasis cannot be placed on the fact that every dollar that is invested by the Government in river improvement in aid of navigation, in great dams to regulate the flow of streams in aid of navigation, in power plants for the manufacture of electric energy, in irrigation, or in any other public service, is a dollar added to the public debt. Whether the project is wise or foolish, every dollar that it costs adds to the national debt and contributes in proportionate amount to the annual burden of interest.

Exactly the same sound business principles should control the selection of public construction projects which are to render a service that the public must pay for as would prevail in the case of a successful private undertaking of the same kind. Many of the plants now being built appear to be foredoomed to failure because of the adoption of the project without adequate study of the site, or complete and accurate plans, or without a proper economic study to determine whether they could be self-liquidating.

The ultimate cost to the taxpayers is the final cost of the project with all of the auxiliary construction made necessary for its complete use, plus all interest and other fixed charges as long as any part of the cost is not amortised, plus all deficits in operation and maintenance less than total revenues.

This may be made more definite by a consideration of actual cases to show that:

(*a*) Imperfect explorations and preliminary and partly digested plans result in misleading and inaccurate estimates, in large contingent expense due to unforeseen difficulties, or to the need for changes in plan after the beginning of work.

(*b*) The adoption of plans for a single project which may take years to build (such as a great dam and power house), involves the building of auxiliary structures to permit its complete use, such, for example, as the transmission lines and sub-stations needed to permit the power plant to serve the market.

These structures must inevitably be built, but no estimates of their cost have been made public by Federal agencies.

(*c*) Where the economic studies of the market and of probable future growth of business were over-optimistic or just plain "bad," the resulting business losses extending over many years, plus fixed charges, constitute an intolerable burden.

These conditions have occurred in the past on numerous occasions when the Congress was misled or was at fault by reason of political pressure. A large part of the public works now in progress (1938) were hastily approved by temporary emergency boards and commissions, with a changing personnel, acting under a pressure not unlike that during the World War when hundreds of millions of dollars were wasted in building nitrate plants that never operated and ships that never sailed. The magnitude of these projects far exceeds anything ever undertaken in a like period of time in the United States.

These hazards of uneconomic public works, construction cost that cannot be repaid, great expense of auxiliary plant not now disclosed, fixed charges that cannot be carried by the project and that may never be reported in its operation, and possible operating losses, cannot be cured by the "writing off" of part or all of the capital account. This has been done in the past, and on the books the account of the particular project looks better. That, however, retires no bonds; nor does it reduce the taxpayers' annual burden of interest. It merely transfers the item to other general accounts. Every project which involves Government in business and which is built as a self-liquidating enterprise, should be compelled by the Congress to keep its accounting records so as to show every transaction, and to carry as its share of fixed charges every dollar that the nation is obligated for by reason of its construction. Failure to treat interest as a deduction from gross revenue is by no means rare in the accounting of publicly owned plants, especially those that are not successful.

Significance of Contingent Expense

A study of the annual reports of the Chief of Engineers of the U. S. Army discloses the fact that Bonneville Dam affords an excellent example of contingent expense due to the hasty adoption of sketch plans based on only a preliminary investigation of the site.

Under the National Industrial Recovery Act the sum of $20 250 000 was allotted to the project on September 30, 1933. Actual construction began in October, 1933. Confident that the preliminary plans were sound, construction work was in progress at the same time that surveys and explorations for the final plans were being made. The details of the preliminary estimate of the dam and power house with two units, totaling $31 000 000, were published in 1934.[41] The PWA allotted an additional $11 000 000 to this project in August, 1934, making a total allotment of $31 250 000.

Detailed study of the site as work progressed indicated that the spillway section of the dam (north of the island) should be moved down stream about

[41] *Engineering News-Record*, November 1, 1934.

3 000 ft. This was done and contracts for the dam and power house were let. In the report of the Chief of Engineers for 1934 the estimated cost was placed at $40 000 000.

Salmon fishing is a most important industry in the area, and public clamor was such that most elaborate fishways and fish locks were installed. The total cost of these provisions for the passage of fish in the river approximates $4 000 000. The Chief of Engineers' report for 1935 gives "the estimated cost of the dam, lock, two power units, and fishways as $42 415 700." The navigation lock was changed from 350 ft to 500 ft in length, and from a depth of 15 ft over the sill to 27 ft. This change cost approximately $1 200 000.

The Chief of Engineers' report for 1935 stated[42] that the project was "adopted by the River and Harbor Act of Aug. 30, 1935." In other words, Congressional approval was not secured until two years after the work was begun. The Annual Report for 1936 states that the estimated cost revised in that year is $51 000 000.

These facts are not presented in any spirit of criticism of the Corps of Engineers, which is doing a magnificent piece of construction; nor is it implied that Bonneville is not economically justified. The writer believes that if there is any warrant for this class of Federal expenditures, Bonneville can be classed as a self-liquidating project. The subject is discussed to illustrate the fact that when such a difficult project as one of these Columbia River dams is selected without the benefit of complete explorations and preliminary plans, and construction is begun immediately, the inevitable result is a great contingent expense the cost of which must be provided for by a further appropriation of funds raised by taxation. In this case the Administrators of the National Recovery Act and the PWA allocated $31 000 000. Two years later, the orphan was "adopted" and Rivers and Harbors appropriations have had to take care of the additional $20 000 000 which was not anticipated.

Estimates made by the U. S. Corps of Engineers include liberal allowances for engineering, interest not only during the period of actual construction but up to the time when the plant is on an earning basis, and all legitimate over-head costs. Contingent costs due to changes of plan, difficult foundations not disclosed by the preliminary survey, weather, flood, or bad soil conditions may increase the final cost of the work from 10% to 50%, or more, beyond the estimate.

Here are two Army built projects so nearly completed that the difference between the 1934 estimate and the revised estimate of 1936 may be accepted as the cost of contingencies:

Dam	1934 Estimate	Revised Estimate of Final Cost
Fort Peck	$86 000 000[43]	$108 600 000[26]
Bonneville	31 250 000[44]	51 000 000[45]
Total	$117 250 000	$159 600 000

[42] Rept. of Chf. of Engrs., U. S. Army, 1935, p. 1315.
[43] *Loc. cit.*, 1934, pp. 641, 742.
[44] *Loc. cit.*, pp. 1335, 1337.
[45] *Loc. cit.*, 1936, p. 1517.

The excess cost over the originally stated estimate is $42 350 000, or an over-run of 36 per cent. If the final cost of Grand Coulee should exceed the estimate of the Corps of Engineers, made under exactly the same conditions as those for Bonneville and Fort Peck, by the same percentage, it means the addition of $157 394 000 of unforeseen cost, or almost as much as the total final cost of Fort Peck and Bonneville. Any one who has visited the site of Grand Coulee in the remote desert, with a wide, deep, and swift river to contend with, whose maximum flood is 300 000 cu ft per sec in excess of the mean flow, can realize that such a contingent expense is entirely possible. These data are based on estimates of the Corps of Engineers, a body capable of making fully as accurate estimates as any other governmental agency or, indeed, as any engineers. The reports of the Chief of Engineers give facts in such full detail as to inspire absolute confidence.

The fact seems to be that the data issued at Washington as the estimates of cost of these major public works projects do not represent the final cost, and that tens of millions of dollars more will have to be provided to complete the work now under way.

Supplementary Costs

Auxiliary construction made necessary by the completion of Federal emergency projects, of which no mention was made at the time of approval of the work, constitutes another great source of future expense.

On December 10, 1935, a Committee on Power of the Oregon State Planning Commission, of which J. C. Stevens, M. Am. Soc. C. E., was Chairman, submitted a report on the "Cost of Bonneville Power." This Committee estimated the capital cost of a transmission system, including receiving sub-stations, for the adequate distribution of power from the finally completed Bonneville plant with ten generating units to Oregon communities, at $43 270 000.

The report of the Division Engineer of the War Department, North Pacific Division, on the study of the Columbia River,[46] refers to this subject of power transmission. No attempt is made to submit any final plan, or total estimate of cost of transmission lines and transformer stations needed to carry the Grand Coulee power to market.

Appendix II of this report includes a study of this problem by Dean Edgar A. Loew, of Washington University.[47] Data in this estimate give the cost of one double-circuit, 220 000-volt line, with its terminal equipment estimated for several circuit lengths, using Government construction costs with interest at 4%, as shown in Table 4.

What has been stated regarding the capacity of these two plants makes it very obvious that the Federal Government has an extremely serious marketing problem, and a very costly auxiliary construction project ahead of it to get the more than 7 000 000 000 kw-hr from Grand Coulee Dam to any possible market.

[46] H. R. Doc. No. 103, 73d Cong., 1st Session, pp. 48–49.

[47] *Loc. cit.*, pp. 456–479.

This same problem of transmission of power will be found at every one of the Federal power projects and will call for tens of millions of dollars in nearly every case. Not only are there no definite figures as to the cost of the dams and

TABLE 4.—COST OF ONE DOUBLE-CIRCUIT, 220 000-VOLT TRANSMISSION LINE

Length of line, in miles	Peak capacity, receiver end	Investment	Annual expense of operation and maintenance	Annual fixed charges*
100	325 000	$8 078 200	$30 000	$551 700
150	271 000	8 495 700	45 000	580 300
200	216 000	9 144 400	60 000	624 600
250	162 000	9 933 600	75 000	678 500

* Fixed charges include interest at 4%; depreciation reserve at 1.78%; and amortization at 1.05%; total, 6.83 per cent.

power plants, but it is impossible to determine for some years the number of millions, or hundreds of millions, of dollars that must be incurred in transmission costs to which the projects are committed.

OPERATING LOSSES UNDER GOVERNMENT OWNERSHIP

The third hazard which has been referred to is that of operating losses on unsuccessful projects. This can best be illustrated with certainty by turning to the history of operating projects.

The experience of the Federal Government in business does not lead one to be optimistic about many of the projects now under construction. Several of the States have engaged in enterprises that have proved disastrous. One or two notable exceptions tend to emphasize the fact that the more far flung the enterprise and the more centralized the administration of it the greater is the certainty of failure.

Municipal ownership, being nearer home, where the taxpayers are neighbors of the officials, has done better. There are scores of examples of well managed and profitable publicly owned water-works. There are several notable examples of highly successful and well operated municipal electric plants in large cities, giving good service at low rates and full and fine publicity to statistics based on accounting that is done in accordance with the most approved classifications. There are many such plants in smaller cities that were built as publicly owned properties in the early days of the industry, which have been, for many years, in charge of non-political commissions and skilled managers, and which compare favorably in service and rates with privately owned utilities of comparable size. On the other hand, there are apparently more than a thousand small-town electric plants that are miserable failures—many more failures than there are good plants.[48]

Early State expenditures for canals were great and there were many projects none of which paid expenses for more than a brief period and none of which ever paid much, if any, interest, to say nothing of any return of capital. The principal surviving canal, the New York Barge Canal, successor to the Erie Canal, had cost the State of New York nearly $300 000 000 from 1903

[48] "Do Municipal Electric Plants Really Pay?" by H. E. Riggs, *Public Utilities Fortnightly*, September 24, 1936, pp. 363–392.

to 1933. At the time that the Barge Canal System was authorized in 1903, State funds of New York had already been drawn upon to the amount of $73 000 000 in excess of canal revenues for the support of the old Erie Canal. In 1933, a total of 4 000 000 tons of freight was moved, and the annual maintenance and operating expense was $2 788 000.

Prior to the Civil War and extending into the 1880's several of the States expended large sums of money on railroad construction, nearly all of which was lost. Only the State of Georgia with its Western and Atlantic Railroad can report a successful investment. This road, still owned by the State, has been leased for more than fifty years at a rental which pays a fair return on its cost.

The Alaska Railroad cost the Government $70 110 000 to June 30, 1930.[49] Not in any one year since it began operating in 1916 has it paid its operating expenses. Added capital investment and interest on the capital have all been met by the taxpayers, in addition to making up the deficits. This property is 540 miles long, serving a population in 1930 of less than 8 400 in a tributary area of 50 000 sq miles, with very little export freight. In the fiscal year 1930 it moved 915 freight car loads.[50] The Government has spent more than $6 000 000 building a highway from Valdez to Fairbanks. In the report of the Senate Committee, the late Hon. Benjamin F. Howell, U. S. Senator from New Jersey, stated:[51]

"These facts afford a notion of the competitive potentialities of the government built and maintained highway in juxtaposition with the also government built and maintained Alaska Railroad, and it well might be urged that these facts also denote, at least, an apparent lack of coordination in the practical promotion of government activities."

In this same report, the following statement appears: "A factor, other than inadequate passenger and freight rates, that contributes to the annual deficits is looseness and inefficiency in the conduct of railroad business and affairs and of the activities incidental thereto."

An examination of the accounts of the Alaska Railroad was reported[52] to the Interstate Commerce Commission on August 31, 1930, by Examiner Leroy S. Price, which is of interest. He reveals the fact that there had never been an audit of the accounts of the Alaska Railroad other than "intermittent and irregular audits of cash in the hands of the disbursing clerk at Anchorage," and finally concludes, "not until the accounts are stated at their true value can the government know the cash outlay in the Alaska Railroad."

The losses assumed by the Government on account of the Federal operation of the railroads during the World War are estimated at $1 120 000 000. Losses due to the Emergency Fleet Corporation appear to have exceeded $2 000 000 000. The operating losses of the United States Shipping Board for the eight years, 1922 to 1929, were in excess of $220 000 000.

The Muscle Shoals Plant on the Tennessee River cost more than $127 000 000, of which approximately $40 000 000 was spent on electric power development.

[49] Senate Rept. 1230, 71st Cong., 3d Session.

[50] *Loc. cit.*, p. 3, Second paragraph.

[51] *Loc. cit.*, 71st Cong., 3d Session, p. 6.

[52] *Loc. cit.*, p. 20.

The project was built primarily for the manufacture of nitrates. The nitrate plants were not completed until the war was ended. The first plant proved impractical; the second was given a trial run of two weeks in 1919 after which it was closed and held as a stand-by plant. Obsolescence has probably been very great on a plant built for such a special purpose as this one, in the eighteen years in which it has stood idle.

The Inland Waterways Corporation is owned and controlled by the United States through the Secretary of the Treasury in whose name all the stock was subscribed. This Corporation operates the Federal Barge Lines on the Mississippi, Illinois, and Missouri Rivers, and from New Orleans, La., to Bingamport, Ala., *via* Mississippi Sound and Black and Warrior Rivers, and through stock ownership controls a railroad known as the Warrior River Terminal Company.

From 1924 to 1935 the system as a whole shows an average tonnage moved of approximately 1 500 000 per year. During the twelve years the system net revenue from operations was only \$1 003 000, and the net income, \$307 865. The total investment in real property and equipment used exclusively by this Government agency in transportation is \$23 500 000. There appear to have been adequate allowances for depreciation of floating and harbor equipment in all years. It will be noted that there is not enough return reported over the twelve-year period to pay even a low rate of interest for a single year on plant actually used in transportation, and nothing at all in the way of recompense to the Government for work done on river and harbor improvement. The property pays practically no taxes and nothing appears as a deduction from earnings for interest on the investment in property and plant used. Had such a deduction been made to represent the actual interest paid by the Government and passed on to the taxpayers, net income would be shown between \$6 000 000 and \$7 000 000 "in the red." The story of this enterprise, in detail, furnishes material for an interesting paper.

The Test of Engineering Economics

The foregoing partial roll call of Federal and State projects in which Government has entered into business especially in times of emergency, causes one who has studied the statistical history of the projects to be decidedly pessimistic as to the outcome of the many and large projects which the Government has undertaken during the past four years.

It will probably be argued that private ownership has had many failures. That is true. There have been hundreds of cases of ill-advised, poorly planned, and badly or dishonestly managed private business undertakings that have resulted in complete failure and the loss of the entire investment. There is a long and unenviable record of privately owned railroads and utilities in which all thought of service was lost, and financial gain alone was the objective of promoters. There are chapters in the history of the railroads, of irrigation, or water-works, and of the electric industry that are scandalous and that gave ample cause for the development of regulation. The answer is that all the money that was lost in the failure of privately owned projects was the money of the individual investors, and the evils of management can be, and should be, cured by Government regulation. Just as soon as uniformity of accounting

and full publicity of accounting and statistics were required of the railroads and real power was given to the Commissions, all the abuses and evils of a generation ago were abolished.

Regulation of private enterprise to prevent improper financing, discrimination of all kinds, rebating, or giving of subsidies, and excessive charges or deficient service is a true and legitimate function of Government that has proved its worth in the last fifty years.

One issue that is clearly raised by the great construction program of the present day is that of national policy. Should the people of the United States continue to favor individual initiative, enterprise, and opportunity regulated by law, or should they adopt a policy of nationalization of utilities and ultimately bring all industry and labor under the control of Government? That is for the nation to decide.

Another question that must be answered deals with the character of public work which should properly be undertaken in an emergency, such as that of 1933 which amounted almost to national paralysis. Millions of men were out of work. One of the great problems was to find work which, promptly and quickly, would furnish labor for the greatest number. Does the adoption of such projects as Bonneville, Grand Coulee, Tennessee Valley, and the many other long-time projects (which require from four to eight or ten years to complete) solve the emergency employment requirement? Or would not just as much employment have been furnished, and just as much steel, cement, and other structural material have been used had there been adopted some such a project as the building of safe and adequate bridges and railroad grade separations on all Federal highways.

In one case there is the planning of a single great structure and the organization of a single force of men for long-time employment. In the other case there is the planning of many structures in each State and the organization of many different construction forces in all parts of the country under the supervision of established highway commissions.

In one case the taxpayers of the nation pay in the end for the entire cost of some of the projects. In the other case, they pay it all, but they would get the lasting benefit of work, which in some of the States would be delayed for years, but which is a direct benefit to public transportation.

The writer believes that in passing upon whether or not any of these great projects are economically wise each one must be judged on its own merits, and all argument that it was an emergency undertaking must be disregarded.

Passamaquoddy certainly cannot qualify as an economic project. There is no demand for power at that location. If there were such a demand the economic answer would be to build a steam plant for \$6 000 000 or \$7 000 000 instead of the proposed tidal plant for ten times as much.

The extensive evidence that was secured by the U. S. Corps of Engineers in its study of the Columbia River seems to show that the bringing into cultivation of more than a million acres of additional farm land is not a present urgent need, that it could only be justified by the sale of vastly more power than can be absorbed by the territory for several decades in order to subsidize

irrigation, and that there exists an engineering report to the effect that the area in question can be irrigated by gravity for less money.

Bonneville, were it alone being built, appears to be a reasonable project with good prospects of being self-liquidating. The one great question involved in that case is that of national policy; and the precedent of the Hoover Dam, designed and commenced during a Republican Administration, cannot be overlooked in considering it.

The policy of Federal improvement of waterways is of long standing. No one can question the propriety of spending money for navigation on streams such as the Ohio and Lower Mississippi which have been main arteries of traffic over the life of the nation; but to spend a quarter of a billion dollars on such a river as the Missouri which has no traffic, and has never had any worth the name, raises another question.

Can the nation afford to increase its burden of national debt at a time when every effort should be made to eliminate unnecessary expense and balance the budget, and in addition increase its annual expenditure for fixed charges, maintenance, and operation by more than $10 000 000 per yr, in the hope that a future water-borne traffic can be developed?

The writer thinks that the answer is "No," but wishes to remind readers that this problem was created by the Republican party.

The entire discussion comes down to a question of engineering economics. Putting to one side all argument as to whether Government ownership is wise or unsound, it would appear to the writer that the general policy that should control the adoption of any program of public works which involves the entry of the Government into any form of business or public service, whether in time of emergency, or not, should be determined by the Congress only, and not delegated to any bureau or commission for final decision.

No such project can be economically sound unless it can earn all operating expenses, including: (1) All cost of maintenance and upkeep of plant; (2) a reasonable allowance for a reserve to replace structures and equipment as they wear out; (3) interest on every dollar of the national debt incurred by the construction of the plant; and (4) a reasonable allowance for amortizing that debt over a period of forty to sixty years. The determination of these facts is an engineering problem. Therefore, no project should be undertaken without an exhaustive study of construction features, and market for the output, by qualified engineers reporting to the Congress. No better example of such a study can be found than that on the Columbia River made by the Corps of Engineers after four years of study.

Every such report should be a matter of record available to the public. There must be scores of reports of highly qualified engineering commissions, such as, for example, the PWA reports on Passamaquoddy, the estimate and recommendation of the Corps of Engineers, and the later report of the Special Engineering Board on the same project, on file in Washington, none of which has been made available to the public.

Allocation of funds was made for the Grand Coulee project in the face of adverse reports by the Chief of Engineers of the U. S. Army, the Board of Engineers of Rivers and Harbors, the Division Engineer, and the bitterly adverse

argument of the Secretary of Agriculture. This was done without any guaranty of revenue notwithstanding the fact that the Chief Engineer of the Bureau of Reclamation, under date of January 7, 1932, submitted to the Commissioner of Reclamation the following perfectly definite recommendation:[53]

"9. No construction on the Columbia River Dam and power plant should be undertaken until the contracts are executed for the sale of power which will insure sufficient revenue for annual expenses and the repayment of the investment in the dam and power plant with interest at 4 per cent within fifty years.

"10. No construction on the irrigation development should be undertaken until the power revenues are assured and a suitable contract for the repayment of the investment in irrigation works within 40 years had been executed."

Every railroad, and practically every privately owned public utility, is required to keep all its accounts under rigidly prescribed classifications issued by Federal or State commissions, and to keep accurately and make public all pertinent statistics. This is as it should be. This requirement has bared many bad practices and made their abolition possible. Unfortunately, no such requirement applies to publicly owned utilities, either Federal or municipal.

In a recent article Arthur E. Morgan, M. Am. Soc. C. E., states:[54]

"Under any method there are certain proprieties and decencies of government which should be observed. Where the public has invited private capital to supply an essential public service, there should be no capricious arbitrariness in destruction of duplication of facilities to the loss of honest, necessary and useful investment. Where bad government or lax administration has allowed inflated securities to be sold to innocent investors the public is not without responsibility. In case public power is used as a 'yardstick' or as a measure of what the private power industry should charge for its services, then it is imperative that records and accounts be honest and fair and open, and that there be no hidden element of subsidy. The very fundamental element of such comparison is honesty, fairness and openness in measurement. Take away those characteristics, and the supposed comparison may becloud the issue rather than clarify it."

It does not appear unreasonable to insist that every publicly owned utility be required to use exactly the same accounting rules and statistical data that are required of similar privately owned utilities. This is of as much importance to the Congress and the Administration as it is to the public.

Conclusions

Twenty-four hundred years ago Pericles made the statement that "acts are foredoomed to failure that are undertaken without discussion." That is just as true to-day as it was before the Christian era.

Who should do the discussing? The Congress, in American Government, is vested with the authority to represent the taxpayers and decide which expenditures are wise and which are not.

Disregarding all the earlier discussions of the Panama Canal and Nicaragua Canal routes between 1870 and 1875 prior to the French activities, the Congress began active discussion of the Panama Canal in 1899, and in March,

[53] Rept. of U. S. Bureau of Reclamation, H. R. Doc. No. 103, 73d Cong., 2d Session, pp. 484–485.

[54] "Public Ownership of Power," by Arthur E. Morgan, *Atlantic Monthly*, September, 1937, p. 342.

1903, passed the Act which authorized the President to negotiate with the Republic of Panama. Not until December, 1903, were negotiations completed. Then came the long discussion as to whether a lock canal or a sea-level canal should be built, ending with the adoption by Congress in 1906 of the lock canal. The actual construction cost of this project was $336 400 000. When the $40 000 000 paid to the French Company, and the $10 000 000 paid to the Republic of Panama are added, the total cost reached $386 400 000, practically $30 000 000 less than the estimate of the Grand Coulee project.

This great canal had the benefit of years of study by a number of the greatest engineers of the time. There was no undue haste. Great questions were not settled by individuals or bureau heads. The records of the Congress show that all questions of general policy, even to deciding the engineering question as to whether the canal should be a lock canal or a sea-level canal, were submitted to and decided by the Congress. The decision was based on the recommendation of the engineers.

If policy has changed; if individuals or bureaus are to authorize expenditures of many hundreds of millions of dollars without Congressional approval, or are allowed to change contracts by the issuance of departmental orders, thereby changing the entire character and objective of the project, is it not time for members of the Engineering Profession to break silence?

The writer believes that those members of the profession whose hands are not tied, who are free to tell the facts, should do so freely and should discuss the economics involved as well as the technical features of public works.

Conditions in the United States to-day call for "hard-headed" economy, for the careful analysis of every proposal to spend money raised by taxation, and for the firm rejection of every one that is not economically sound.

Members of the Engineering Profession, better perhaps than any one else, can advise the people of the United States as to the economic soundness of proposals to spend vast sums on public works, especially those designed to render public service of a kind that should make the project self-liquidating.

They will fail in their duty if they keep silence.

AN APPEAL TO REASON

By William J. Wilgus,[55] Hon. M. Am. Soc. C. E.

Synopsis

In this paper, the writer wishes to avoid the impression that he disbelieves in the creation of worth-while public works of the type considered in this Symposium. In the words of the Hon. George D. Aiken,[56] Governor of Vermont:

"I hold no brief for any private utility corporation. I even hold that through their manipulations in pyramiding values [too often fraudulent] and the acquiring of unearned and unwarranted profits at the consumer's expense, they are largely, if not principally, responsible for the present state of affairs."

The bracketed comment is by the writer. Neither does the writer hold with those who would maintain the unemployed in corrosive idleness, destructive of all that is best in American life, rather than at self-respecting work, at a somewhat greater expense, from which the public gains material and spiritual returns. The work, in fact, must be worth while in the public interest and not uneconomic and wasteful, involving a tragic loss of public monies for ever-continuing interest charges and upkeep.

Worth-while public works need not necessarily suffer from the evils of corrupt, incompetent political management. There are notable examples of admirably run publicly owned water-works, highly successful and well operated municipal electric plants, toll tunnels and bridges and the other commercial activities, such as those of the Port of New York Authority, the revenue-producing enterprises operated under the auspices of the New York City Park Department, and various toll-bridge authorities in New York City, and the publicly owned transit facilities in the same municipality which eventually are to be expanded by the addition en masse of its privately owned subways and elevated lines. There are, too, many city managerships to be looked to as outstanding examples of successful administration; this can likewise be said of many works under the jurisdiction of the U. S. Army Engineers. Under proper safeguards and conditions, it is possible to run public enterprises with honesty, competence, and resourcefulness, coupled with a strict observance of proper accounting at all times open to the public. The quicker the people realize this and prepare for it, the better off this country will be. To drift blindly, without a chart or compass for guidance, will mean the continuation of the ills of public management, vastly to be multiplied, as private utilities, whether one likes it or not, come more and more under public control. With the public ownership of railways and other utilities "in the offing," it behooves the civil engineer to abandon a King Canute-like attitude

[55] Cons. Engr., Weathersfield, Vt.

[56] Address at Plymouth, Vt., August 8, 1937, reported in *Rutland* (*Vt.*) *Herald*, August 9, 1937.

of negation and to favor a constructive course leading to a reasonably satisfactory conclusion.

It is deeply to be regretted, therefore, that the gargantuan projects analyzed in this Symposium have been promoted by the Government without at the same time offering evidence that they hold promise, within a reasonable time, of becoming socially and economically justifiable under efficient management.

Major Projects

In the Passamaquoddy Tidal Power Project, the estimates of cost have ranged widely from \$37 000 000 to \$62 000 000, to which must be added unknown sums of magnitude for distribution systems and for interest, and other carrying charges, until the project, if ever, shall become self-supporting. It would also appear that the intended dual methods of operation—by water power and by coal or oil—will result in an absurdly high cost per unit of capacity; and there is no market for the power in sight. Furthermore, it is said that certain engineering reports on the project have been withheld from the public. As so far revealed, it is not a "pretty" story, especially in view of the threat that workers in the coal and oil fields, and in the transportation industry, are to be made to suffer to the extent that water power thus is to be wastefully used.

As to the Fort Peck Dam, the estimate of cost has risen from \$86 000 000 to \$108 000 000, for which the resulting improvement in the low-water flow in the Missouri River will bring a grossly inadequate return to the public. It may be that in some unexplained manner the improved river channel is expected, within a reasonable time, to afford a water outlet for the products of the Northwest at savings in the cost of transportation sufficient to offset the added interest charges, the expense of upkeep to be borne by the people of the country at large; or, it may be that other benefits worth the price are to be expected by the public in the way of flood control, improved sanitation, cheap power, and the reclamation of waste lands. The evidence, however, appears to be lacking.

Likewise, the estimated cost of the Grand Coulee project in the Columbia River Basin has risen from \$108 000 000 to upward of \$437 000 000 for the combined purposes of navigation, power, and irrigation, to which must be added the first cost of needed distributing systems and auxiliary structures of magnitude, as well as the annual costs of depreciation, maintenance, operation, and interest on the capital cost, from the time of completion of the project until it shall perchance become fully self-supporting in the uncertain future. In this, as well as in the case of the Fort Peck Project, the claim is to be borne in mind that already there is a superabundance of irrigated lands in the West needing competent settlers in large numbers to make them an economic success.

Justification of Foresight

In total, the present estimated cost of these three outstanding projects is upward of \$600 000 000, plus the unfigured enormous additional costs due to contingent expense and operation. In judging their merits or demerits from

the economic angle it seems necessary to use caution. Vision—or, perhaps, prophecy—has been the moving cause from which some of the world's most notable products of the brain and the hand of Man have come forth. Witness, for instance, the Canadian Pacific Railway, built by men of vision across plains and mountains then unpeopled and undeveloped, with buffalo bones at first as the only freight in sight. The electrified Grand Central Terminal, in New York, N. Y., may also be pointed to as an example of imagination come true, with its rich by-products gained through the utilization of air rights, initiated in the face of forebodings that offices in that region could not be rented in competition with the established down-town Wall Street district; that guests would not patronize hotels and apartments over or neighboring a smoky, dingy, noisy railroad station; that cabs, then horsedrawn, with their attendant odors, could not be parked underground; and that vehicular travel could not be diverted from Fifth Avenue to Madison and Park Avenues, nor to the intended elevated driveways around the station. A vision of the future, bearing directly on this subject, may be quoted from a report issued,[57] in 1937, by the National Resources Committee:

"Another material (than manganese) which is essential to the production of alloy steels, as well as for a coating metal, is chromium. It also is largely produced from foreign ores. There is a recently developed Bureau of Mines process for producing ferrochrome electrolytically from chromite ore. There is still considerable developmental work to be done on this, but if it proves feasible it will very likely be a practical outlet for the power to be produced at the Grand Coulee Dam. A new ore deposit of very significant size has recently been opened up in the Philippines and contains about 35 per cent of chromium oxide. It would be feasible to ship this material to the Columbia River water-head for electrolytic recovery of ferrochrome. There are also considerable deposits of this same type of ore in Montana and one county in Wyoming, which would be available in case of need."

Other possible developments in electro-metallurgy in the field of such metals as aluminum, magnesium, manganese, and iron, are pointed to in the same report, as well as the possible creation of new manufacturing centers convenient to sources of raw materials and a consuming public, and the reclamation of waste lands for the purposes of agriculture.

Wishful Thinking versus Foresight

These are only hints of what may "come out of the blue" when new conditions are created, such as improved channels for transportation, changes of motive power, and pools of cheap power of huge proportions accessible to ocean vessels. Wishful thinking or idle guessing should carry no weight in prophesying what new developments may cause the Passamaquoddy, Fort Peck, and Grand Coulee Projects to be economically justifiable. Civil engineers have a right to ask just what line of reasoning in this respect has guided their Government, so that an intelligent conclusion may be reached as to its soundness, and the total carrying charges—interest and upkeep—that necessarily must be borne by the people of the United States before the project in each instance shall, perhaps, become self-supporting.

[57] Rept. on Technological Trends and National Policy, by the National Resources Committee, 1937, p. 357.

In a word, then, there is reason to demand: (1) A defense, if one is possible, of the original committal of the people of the United States to projects based on the promoters' "half-baked" plans in connection with which the estimated costs later have been found to be far too low; (2) an explanation of the excessively high cost per unit of capacity or development that characterizes many of the projects; (3) reasonably accurate estimates of cost inclusive of all elements of every nature; (4) an outline of the reasoning, with data, in support of the alleged economic justification of each of the projects; and (5) a disclosure of the engineering and other reports and opinions hitherto held *en camera* pertaining to some present Federal projects.

Florida Ship Canal

It is not alone to the three major projects at Passamaquoddy Bay, Fort Peck, and Grand Coulee (for knowledge of which the writer is indebted to Professor Riggs' paper) that this paper is confined. The writer has also had occasion to familiarize himself with the Government's Florida Ship Canal project, which, in many of its aspects, is even more forbidding.

Basic Considerations.—The Peninsula of Florida, about 95 miles wide at its narrowest point, projects (with its terminating arc of islands) about 440 miles south and southwest to Key West and beyond, bounded by the Atlantic Ocean on its east, and the Gulf of Mexico on its west. The Straits of Florida, on its south and southeast, extend about 250 miles to the Island of Cuba and 40 miles to the Bahama Banks, with a minimum depth of 1 000 ft. Generally low in altitude, the backbone or central axis of Florida, for a width of 40 to 50 miles, rises to elevations of from 100 to 200 ft above sea level with a few heights or knobs rising to 300 ft and one to an elevation of 325 ft.

Underlying the Peninsula is the porous Ocala limestone formation, honeycombed with caverns and intricate channels of unknown location, number, and size, and dipping from north to south to levels far below sea level. On the flanks of this rock formation, to a greater or less degree, a comparatively impervious material is superimposed except at two eroded areas on the backbone of the State which act as catchments for the rain that there descends into and permeates the limestone cavities and, as fresh water, flows upward under pressure through artesian wells and at breaks at lakes and springs, and in the open sea. The northerly of these eroded areas, roughly 50 miles wide and 120 miles long, is separated from the one on the south by a low-lying saddle, about 100 ft above the sea, centering at Ocala, Fla., about 75 miles below the latitude of Jacksonville, Fla., on the Saint Johns River. Draining eastward from this saddle are the waters of the Oklawaha River emptying into the Saint Johns River near Palatka, Fla., flowing northward and eastward to the Atlantic Ocean, and westward the waters of the Withlacoochee River, emptying into the Gulf about 94 miles north of Tampa Bay. The fresh water in profusion thus under pressure beneath the Peninsula is vital to the culture of citrus fruits and vegetables, and the attraction of visitors from the North, on which the people of the State depend for their livelihood. This fact, as will be seen, is first to be borne in mind in giving study to the Florida Ship Canal problem.

Along with this is to be viewed the effect of the Peninsula on the time consumed by water-borne commerce in passing between the Gulf of Mexico and the Atlantic Coast, and foreign countries, through the Straits of Florida. Since the number has steadily increased in recent years, it is estimated that in 1935 as many as 12 000 vessels carrying more than 60 000 000 short tons were engaged in this service, principally in the movement of petroleum products, of which about 80% or 90%, it is variously claimed, is to be considered in weighing the pros and cons of a short-cut across the State.

History of Engineering Analyses.—In the 1920's the growing desire to shorten the journey between the Gulf of Mexico and the Atlantic led to various surveys by the Corps of Engineers, U. S. Army, for a barge canal connecting the Atlantic and Gulf intra-coastal protected waterways, and in the early years of the present decade for that as well as an alternate ship canal. A series of reports then ensued, eight in all, in which the 9-ft barge canal project was ignored or disapproved, and the ship canal of differing designs and estimates was in some instances approved and in others disapproved.

In all these reports the adopted general course of the proposed ship canal has been the same, running, as it does, westerly and southerly in the Saint Johns River from deep water off its mouth in the Atlantic Ocean for about 78 miles, past Jacksonville, to Palatka; thence, southwesterly and westerly in the valleys of the Oklawaha and Withlacoochee Rivers to the Gulf coast, in a 95-mile artificial cut of earth and rock having a maximum depth of 141 ft or more as recommended in the later reports; and thence, for from 25 to 27 miles in the rock-underlaid shoal waters of the Gulf to deep water, a total length of nearly 200 miles. Thus would be provided, it is claimed,[58] "a route between the Gulf and the Atlantic seaboard from 300 to 350 nautical miles shorter than the present route around its [Florida's] southern end," and a saving of "somewhat less than one day's steaming time."

It is the notching of the lip of the ground-water reservoir by this deep cut at the edge of the northerly catchment area to which reference has been made, that has given rise to the fear that the resultant lowering of the head in artesian wells and springs in many parts of the State will permit salt water to rise and cause damage to fruit and vegetable culture and seriously affect lake levels and municipal water supplies.

The first of these reports, a preliminary one made at the instance of the President by a Special Board of Army Engineers, June 3, 1933, was intended for the use of the engineers of the Reconstruction Finance Corporation and was not made public. The recommendation in a second report, made by the engineers of the Public Works Administration (PWA), October 19, 1933 (in connection with an application of the Ship Canal Authority of the State of Florida for a public loan of $115 000 000 to cover the estimated cost of a toll 30-ft lock canal), was not approved by the PWA Administrator, and failed of action.

The third report, made by a Special Board of Army Engineers, December 30, 1933, placed the cost of a 35-ft lock canal, with a minimum channel width

[58] Committee Print on "Atlantic-Gulf Ship Canal, Fla.," 75th Cong., 1st Session, pp. 2, 3, and 28, respectively.

of 250 ft, at $199 481 000 plus $8 562 000 for land and damages, a total of $208 043 000, exclusive of interest charges during construction. This report, at the request of local interests favorable to the enterprise, was not made public until three years later, after work had been started on the canal, when it developed that its verdict had been adverse to the economic justification of the project at this time, as had been the findings of the Shipping Industry contained in a report made by the Department of Commerce on February 1, 1934.

The fourth and fifth reports were made by the President's Interdepartmental Board of Review on June 28 and September 15, 1934, respectively, which estimated the cost of a 30-ft sea-level canal, with a minimum channel width of 250 ft, at $142 700 000, exclusive of interest charges, and $159 834 000, inclusive of interest, plus $3 000 000 for rights of way to be borne by local interests. The project was declared to be not self-liquidating by tolls collected from shipping, the annual charges being placed at $7 250 000 and the estimated benefits at $7 500 000, which was too close to finance the project on that basis. In the following January the PWA reached the same adverse conclusion as had been the verdict contained in the report of the Department of Commerce a year before. Nevertheless, the President, on August 30, 1935, allocated $5 400 000 from the Emergency Relief Fund for starting the work, in addition to which money for acquiring the necessary rights of way was provided by the Florida Ship Canal Navigation District, created by the State Legislature. Apparently, from then on the project was to be considered as toll free to all users, foreign and domestic.

The sixth report was made by the Revisory Board of Army Engineers, on November 1, 1936, after the relief allotment had been exhausted, in which the estimated cost of a 33-ft sea-level canal, with a minimum channel width of 250 ft, was placed at $162 985 000, exclusive of interest charges, or $185 471 000, inclusive of interest charges, and of $3 000 000 to be borne by local interests for rights of way, damages, etc. The project was declared to be economically justifiable, a verdict arrived at by calculating annual savings in ship-operating expenses, interest on ship cargo, freight rates, and navigation hazards plus recreational value to pleasure craft at $9 553 244, and annual net carrying charges at $8 242 969, leaving an alleged net annual gain of $1 310 275, which was held to show that the project was economically justifiable on a toll-free basis. The latter figure, however, would not in its entirety accrue to American interests. In the estimated savings ($9 553 244) is included the value of savings in ship time and interest on cargo ($7 155 720) of which 20% (amounting to $1 431 144) should be enjoyed by vessels flying foreign flags. This would leave annual savings of only $8 122 100 for the benefit of American citizens, in contrast with the annual carrying charges amounting to $8 242 969. On this showing the project is not economically justifiable, even granting that the benefits flowing to private interests in some manner really would be passed on to the public called upon to pay the bill.

Then came the long deferred hearings before the Board of Engineers for Rivers and Harbors late in December, 1936, without which in the usual course the project could not be considered by Congress for approval or disapproval.

This resulted in the report of the Board on February 24, 1937—the seventh in this series—fixing the estimated cost of a 35-ft sea-level canal, in preference to a shallower one, with a minimum channel width of 400 ft, at the sum of \$263 838 000, exclusive of interest charges (computed at \$25 005 000), and exclusive of \$3 000 000 for right of way to be paid for by local interests. The total cost to all interests, therefore, was placed at \$291 843 000, in which provision is made for the removal of nearly a billion cubic yards of earth and rock. In this estimate, as in all the others, nothing was included for consequential damages resulting from disturbances of the underground water supply. As bearing on this the Board stated:

"In view of the undeterminable possibility that the excavation of a sea-level canal might open underground channels in the Ocala limestone of such size and extent as would drain a wide area, with consequent extensive damage to ground-water supplies, any authorization for the construction of the canal should empower the Secretary of War, on the recommendation of the Chief of Engineers, to modify the plans to provide for the construction of a lock canal if at any time during the progress of the work such a modification should be found warranted in the interest of protecting the ground-water supply."

No estimate, in such an event, was named for the additional cost of making the change, nor for claims for indirect damages which, according to the Board, the Federal Government under the law would face the onus of meeting. The Board declared that the benefits to be reaped from such a canal "do not establish economic justification for the large expenditures necessary for its construction," this conclusion having been predicated on estimated annual charges of \$14 930 000 and benefits or savings of \$8 543 000, leaving an annual net loss of \$6 387 000 apart from diversions of benefits to vessels flying foreign flags.

Then came an eighth report—that of the Chief of Engineers on April 1, 1937—disagreeing in large part with the recommendations of the Board of Engineers for Rivers and Harbors. The Board estimated the cost of a 33-ft sea-level canal, with a 400-ft minimum width of channel, at \$197 921 000, apparently exclusive of interest charges (or \$215 735 000 including them), plus \$3 000 000 for right of way to be borne by local interests, a total of \$218 735 000 in contrast with the aforementioned Board's estimate of \$291 843 600 for corresponding items applicable to a canal 2 ft deeper required, in its opinion, for safety. The Chief of Engineers does not "share the apprehension expressed in the report of the Board of Engineers, as to the possible adverse effect [of a sea-level canal] on ground-water supplies." He places the annual charges against the United States and Florida at \$8 905 000 and the benefits to shipping at \$8 741 000, or practically a stand-off of what the writer considers private gains at public expense. Making allowance for the portion of these benefits that would flow to vessels flying foreign flags, the result would be even more painful to the people of the United States.

Conclusions from Available Data.—At this point it should be remarked that, for some mysterious reason, the item of interest charges during construction, in the Government estimates, is treated as a thing apart, to be ignored in giving the public an impression of the total cost of the project. Hire of

money, termed interest, is, of course, just as much an element of cost to be borne by the taxpayer, as hire of labor or costs of material. To omit it from the total estimates is in this respect to misguide the public.

It will be seen that the estimated costs of the project, to the Federal Government and Florida "interests," have varied from a low of $115 000 000 to a recent high of nearly $300 000 000, equal in amount per ship-mile saved to many times that incurred at the Panama and Suez Canals, to which is to be added the cost of damage arising from "possible" injury to the ground-waters of Florida and the added cost incident to the changing of the canal from sea-level to a lock design in case the later discovered certainty of such injury would make that course necessary.

Whether or not the net benefits to be gained from this vast and uncertain expenditure make the enterprise justifiable from a self-liquidating or economic standpoint has been open to a variety of opinions. The answer is "yes" on the part of the Revisory Board in 1936 and the Chief of Engineers in 1937. It is "no" on the part of the Special Board of Army Engineers in 1933, the shipping industry as reported by the Department of Commerce in 1934, the President's Interdepartmental Board of Review in 1934, the Administrator of PWA in 1935, and the Board of Engineers for Rivers and Harbors in 1937.

Such a jug-handled arrangement whereby the beneficiaries of the canal, domestic and foreign, are to be free to keep their savings while the taxpayers —that is the people—"foot the bills" of undefined magnitude, has an even more repugnant look when it is understood that by the time the canal should be completed in six or more years, it would run the risk of being already obsolete and unused.

In viewing this aspect of the situation, it is well to bear in mind that the margin of saving in steaming distance *via* the proposed Florida Canal is very slight in comparison with that enjoyed *via* the world's existing great ship canals. In the case of the Suez Canal, the saving over its Cape of Good Hope rival is, say, 40 times its length of 100 miles; and in the case of the Panama Canal the saving over its Cape Horn and Cape of Good Hope rivals is, say, 100 to 200 times its length of 50 miles. Both are in sharp contrast with the Florida Canal's saving over its Florida Straits rival of less than two times its length of 200 miles. The two cape routes are hopelessly out-distanced by the interoceanic cut-offs with which they have to cope. The Florida Straits route, on the other hand, is not so handicapped and through improvements in vessel design, and a less hazardous course, has every prospect of "holding its own," for reasons on which it may be well to touch.

The estimated saving in ship time through the use of the proposed Florida Canal as against the Straits of Florida (22 hr) is based on a difference of vessel speeds *via* the two routes which will become less and less as progress is made in the art of shipbuilding. The Chairman of the U. S. Maritime Commission recently has stated[59] that "the United States is third in world tonnage (mostly obsolete), fifth in speed of ships, and eighth in ranking of modern ships in operation." With an increase of 50% in the speed capacity of vessels, already said to have been realized in the mercantile fleets of foreign

[59] Reported in *New York Times* in dispatch from Washington, D. C., August 16, 1937.

countries with which the United States must compete, there will be no appreciable saving in time to be made through the use of the proposed shorter canal route in which advantage may not be taken of modern open-sea speeds. Even under present conditions many of the steamship companies now navigating the Straits of Florida assert that, despite its intended absence of tolls, they will not use the canal, if built, because of higher insurance premiums on hulls and cargo passing through the canal, and[58] "the hazard of damage by collision or grounding in the constricted waters of the canal" which the Board of Engineers for Rivers and Harbors considers "would greatly exceed the hazard of damage in the open waters of the Florida Straits including damage and delays due to hurricanes in the latter."

In a word, unlike the Panama and Suez Canals and unlike other artificial waterways having no alternative routes to threaten their use, the proposed Florida Canal would ever face the prospect of the continued use of a competitor having lesser handicaps and the superior advantages of open-sea navigation. The margin of saving in time *via* the new route is too slight to warrant an expenditure upon it of $200 000 000 or $300 000 000, or more, which the people never can "get back," to quote the words of the President who is reported to have named this as a condition.[58]

From what has been stated herein[58] in respect to the toll-free Florida Ship Canal project the situation may be summarized as follows:

(1) The estimated cost of the project has risen in the four years, 1933 to 1937, from $115 000 000 to as high as $291 843 000, with the assurance of a marked increase, of unknown size, should a change be found necessary from a sea-level to a lock design, and should consequential damages be incurred through injury to the underground water supply of Florida;

(2) The estimated carrying charges to be borne in this connection by the people of the United States have risen in the same period from $7 250 000 to as high as $14 930 000 per yr, also with the assurance of a marked increase should the aforementioned conditions prevail;

(3) The estimated benefits to be enjoyed by the users of the canal and its other beneficiaries—foreign and domestic—have ranged from a low of $7 500 000 to a high of $9 553 244 per yr, constituting in effect a public subsidy to private interests from which the people of the United States as a whole would gain nothing whatever, other than the possibility of some lowering of freight charges of unknown extent, of which there is, to say the least, a doubtful chance when competition rather than cost of service is what governs the fixing of rates;

(4) In thus failing so signally to show benefits in excess of carrying charges, the canal project falls far short of meeting the condition laid down by the President that, to have his approval, it should in effect bring to the people of the United States the assurance that they would get their money back;

(5) The possibility of grave injury to the underground water supply of Florida, through the building of a sea-level canal, is one to be viewed with concern by the people of the State and nation. It is comparable to a community, informed that an important bridge on which the people depended for access to the world might fail, with resulting death or injury to their families

—a case in which to doubt is to condemn; where there are no compensating advantages to offset the risks;

(6) The disruption of Florida's highway system along the canal, involving the creation of needed future crossings at the expense of the State, also to be burdened with the inconvenience and cost of upkeep and operation of highway ferries and drawbridges as well as the costs of rights of way and other lands for the canal; the cost of appropriate terminal and transfer facilities; and, the cost of maintenance and operation of the canal in excess of a fixed annual sum to be borne by the Federal Government, should the recommendation of the Revisory Board in this respect be endorsed by Congress, are all matters of great moment to the taxpayers of that State in which, too, sight is not to be lost of threatened injury to its wild life; and,

(7) The modernizing of the country's merchant marine, about to be undertaken, will result in the employment of speedier vessels in the Gulf-Atlantic trade and, in consequence, the continued use of the safer present open-sea route around the tip of Florida as against the canal route then offering no substantial saving in time and greater dangers of navigation, thereby threatening the canal, if built, with early obsolescence, even if the exhaustion of oil fields tributary to the Gulf is long postponed.

Recommendations.—In view of the foregoing conclusions, therefore, the Florida Ship Canal project, as a substitute for, or rival of, the existing open-sea route through the Straits of Florida, is without a single sound argument in its favor, and should not receive the stamp of approval by Congress, accompanied by appropriations for its continuance. It is better thus to abandon what was started as a relief measure at a cost of $5 400 000 of Federal money and an unknown sum thus far expended by local interests for right of way, than to throw good money after bad.

In taking this stand against the financing of useless public works it is not meant, of course, that Federal funds should not be devoted to the construction of really worth-while projects in order to give work to the unemployed. On the contrary, the money saved from loss on uneconomic enterprises like the Florida Ship Canal may, and should, be devoted to the supplying of such crying needs as added public thoroughfares for a rapidly growing automobile traffic. For instance, the building of an International Parkway along the Appalachian Uplift, in co-operation with Canada, connecting many national and State parks and forests and other recreational areas, reaching from Montreal, and Quebec, Que., and the Gaspé Peninsula on the Gulf of St. Lawrence, to Key West, Fla., and other attractions on the Gulf of Mexico, would cost no more in the United States than a Florida Ship Canal; it would give employment to skilled and unskilled labor in seventeen States of the Union instead of one; it would serve to protect the mountain forests and streams from private spoliation and restore many of them to their original charm; and, from a social viewpoint, it would provide the public with a safe, convenient, and inspiring means of movement between the widely varying climates, scenes, customs, and fauna and flora of the North and South, free from commercial usage, billboards, and other objectionable features, in regions neighboring

great industrial centers where utility and pleasure thereby would be sanely served in these days of increased leisure.

A Challenge

The papers of the Symposium constitute a terrific arraignment of those who are entrusted by the people of the United States with the handling of their affairs. Great public works are declared on the record, to have been conceived in error. Can it be that the Engineering Profession is not in possession of all the facts with which to reach a just conclusion? If so, the Government's side should be presented without reserve. As citizens of a democracy, and as engineers qualified for the purpose by education and experience, the Civil Engineering Profession should, in the interest of the public, approach this question in a non-partisan spirit, searching only for the truth. In the words of Professor Riggs, "They will fail in their duty if they keep silence."

In the case of the Florida Ship Canal, as in the instances referred to by other authors of this Symposium, involving in all a possible expenditure of more than a billion dollars, it is to be hoped that their proponents—members of the Society and others—will draw attention to any errors of fact which, unintentionally, may have been introduced herein, and to any reasoning that they may care to offer in rebuttal. To arrive at the truth is the important objective, so that engineers may properly advise their fellow citizens who are not experienced in such matters. Thus will there be ground for hope that the hazards of uneconomic public works construction in the future will be avoided, or at least lessened. In that cause there can be no higher call to public duty.

Acknowledgment

In the preparation of this paper, in addition to other sources cited by footnote references, the writer has made liberal use of the factual data presented by Professor Riggs in this Symposium.

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PAPERS

FLOOD ROUTING

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Synopsis

A method used for routing floods through the Tennessee River is described in this paper. First, a method for routing floods in the natural condition from Knoxville, Tenn., to the mouth of the Tennessee River was developed, and then the procedure was adapted to routing through the reservoirs of authorized and proposed projects. To analyze flood reduction, the complete river system was considered as a unit for various floods.

Routing was accomplished by dividing the river length into seven reaches and applying the storage equation to each reach. Total inflow into each reach was determined day by day from the routed outflow of the adjacent up-stream reach, published discharges of metered streams flowing into the reach, and an inflow estimated from rainfall on the unmeasured areas draining into the reach. Total flood volume at each dam site was assumed to be known. The storage in the reach for various flows was determined from topographic maps and cross-sections of the river valley in conjunction with flow profiles determined from back-water curves. The outflow was then derived from the relation of storage and discharge for the reach.

The general procedure for all reaches is described, and a detailed example for the Watts Bar-Chickamauga Reach is included. Finally, results are given for the effect of the proposed system of dams on the 1926–1927 flood and on a flood 50% larger.

Introduction

The development of the Tennessee Valley by the Tennessee Valley Authority (TVA) involves the construction of a series of reservoirs for navigation and

Note.—Written comments are invited for immediate publication; to ensure publication the last discussion should be submitted **June 15, 1938.**

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flood control, with incidental power development. The program that has been authorized by Congress includes storage reservoirs on two tributaries of the Tennessee River and a series of five high dams on the Tennessee River itself. The ultimate plan for the development of the Tennessee Valley includes the construction of seven high dams on the main river, in addition to the existing Wilson Dam at Muscle Shoals, Ala.

The determination of the reduction in flood crests that may be effected by such a system of reservoirs is a complex problem. It may be a comparatively simple matter to determine how a storage reservoir on a tributary modifies a flood flow, if stream-flow data and the storage capacity of the reservoir are known. However, the total volume of water that passes through the main river within a flood period is so great, with respect to the available storage capacity in the river reservoirs, that much study is necessary for an estimate of the reduction which these reservoirs will afford. Since river reservoirs are not level pools, consideration must be given to the water-surface profiles of each reservoir. In some of the river reservoirs the back-water effect of the dam disappears some distance from the upper end of the pools when there is a large flood discharge. The solution of the problem, therefore, requires a knowledge of the total storage in each pool which comes into use by an increase in water level—at the lower end of the pool due to the dam itself, and in the upper end due to the limitation of channel capacities. In comparing flood flows under "natural" conditions with those under conditions after the dams are built, it is necessary to determine the effect of channel or valley storage on the natural flood flows.

A complete knowledge of conditions during past floods would require copious data on channel cross-sections, topography in the valley reached by the flood, the amount of clearing within the flooded areas, and the actual profiles of the river during the high flows. It is possible to obtain much of this information, but a reliable estimate of the effect of some factors requires considerable research. (The study of floods has involved, first, as thorough an analysis of actual floods of the past as existing data permit; and, second, a determination of the effect of reservoir storage, now or soon to be available, upon floods of the past and others that may be expected in the future.) Certain reservations of storage capacity are made for flood control, but it is necessary not only to determine flood-control benefits which may be derived by the reservoir system, but also to provide data which may be necessary or useful for the proper operation of the system in flood periods.

General Information

The Tennessee River (Fig. 1) has a total length of 652 miles from its mouth, near Paducah, Ky., to the confluence of the French Broad and Holston Rivers, at Knoxville, Tenn. The valley is divided into two basins by a constriction near Chattanooga, Tenn., the upper basin being much more mountainous than the lower. The river has a number of relatively large tributaries, and many small tributary streams. The slope of the main river channel varies from rather slight slopes on the lower river to steep slopes on the upper river and at Muscle Shoals, in Alabama. Along parts of the river there are wide over-bank

sections, as is the case just below Pickwick Landing Dam, in Tennessee, and in other parts there are gorge sections, as is the case just below Chattanooga. For a general description of the Tennessee River Basin, the reader is referred to the March, 1936, report to Congress of the TVA on "The Unified Development of the Tennessee River System."

Tennessee River Survey maps made under the direction of the United States Engineer Department and of silt ranges made by the TVA are important sources of information for use in determining storage in the various reaches. River-stage records and high-water marks and profiles were obtained from both published and unpublished data of the United States Weather Bureau, the United States Geological Survey, the Mississippi River Commission, and the Tennessee River Survey of the U. S. Engineer Department. Later data have

Fig. 1.—Tennessee River and Tributaries, Showing Existing and Proposed Dams

been obtained and compiled directly by the TVA. Rating curves were obtained from the same sources when available; otherwise, they were constructed and extended from gage relationships taken from the river-stage records and from back-water curve computations.

General Procedure

The storage equation, "inflow minus outflow equals change in storage," has been made the basis of the routing. There are a number of solutions of this equation adaptable to flood routing, and the one used is that described by R. D. Goodrich, M. Am. Soc. C. E.[4]

In routing a flood through a reach, it is necessary to know: (1) Total inflow into reach: (*a*) Inflow at upper end of reach; and (*b*) local and tributary inflow; (2) profile of water surface at any instant; and (3) storage under the profile.

[4] "Rapid Calculation for Reservoir Discharge," by R. D. Goodrich, *Civil Engineering*, February, 1931, pp. 417, 418.

If these three factors are known, or can be closely approximated, an outflow hydrograph will be obtained by solving the storage equation,

$$\frac{I_1 + I_2}{2}t - \frac{O_1 + O_2}{2}t = S_2 - S_1$$

in which I is the total inflow rate; O, the outflow rate; S, the volume in storage in the reach; t, the length of the period; and the subscripts, 1 and 2, refer to the beginning and the end of the period, respectively. The average inflow and outflow for a period are $\frac{I_1 + I_2}{2}$ and $\frac{O_1 + O_2}{2}$, respectively. However, if the unit of flow is cubic feet per second and the period is 24 hr, $I_1 + I_2$ and $O_1 + O_2$ give the total inflow and outflow in acre-feet, since 1 cu ft per sec for 24 hr is almost exactly 2 acre-ft. With these units the storage equation can then be written as $I_1 + I_2 - O_1 - O_2 = S_2 - S_1$, and transposing, $O_2 + S_2 = I_1 + I_2 + S_1 - O_1$. For other units and lengths of routing periods the equation must be solved accordingly.

The outflow, O_1, for the first period must be estimated, but can be closely approximated as the routing is started a day before the river begins to rise; any error made in this estimate is soon eliminated. With O_1 assumed for the beginning of the first period, I_1, I_2, S_1, and O_1 are known and determine an "outflow-plus-storage" factor, $O_2 + S_2$, for the first period. Then, with the relation between O and S known for various values of their sum, the separate values of O_2 and S_2 are determined and are equivalent to O_1 and S_1 of the following period. The four quantities, I_1, I_2, S_1, and O_1, are then known for the second period and the process is repeated.

Reaches

The first part of this study consists of routing actual floods in the natural river channel, under conditions existing at the time those floods occurred. As a preliminary, the river must be divided into "reaches" of suitable length.

In the present study of the Tennessee River, a time unit of 24 hr is used. A more flashy stream would require a shorter routing period since, in the solution of the storage equation, it is assumed that changes in rates of inflow and outflow are uniform for the period used. The length of reach should be such that the time of travel through the reach is about the same as the time unit used in routing, so that changes in the condition of inflow at the upper end of the reach at the beginning of a period will be reflected at the lower end of the reach by the end of the period, with proportional intermediate changes. The desirable length of reach in the present case is thus seen to be about one day's travel.

The ideal situation would be to have a "routing station" near the mouth of each large tributary, and as many others as necessary; but it is difficult to fix the stage-discharge relation at such points, and the ratings are seldom available. However, it is desirable to know the natural hydrograph at each dam location so that a comparison may be made between the natural peak discharge and the regulated peak discharge after the dam is built. Moreover, local and tributary inflows that have proved satisfactory in the natural condition will be available for use in routing with the dams built. The routing stations were thus fixed by

the location of the dams whose potential effects on floods it was desired to study. Fortunately, all the reservoirs except Gilbertsville, in Kentucky, are close to the length corresponding to a time of travel equal to one day. Table 1 gives pertinent data for the reaches and dams considered.

TABLE 1.—GENERAL DATA ASSUMED FOR DAMS AND RESERVOIRS

Name of project (1)	Miles above mouth (2)	Drainage area, in square miles (3)	Surcharge elevation for floods (4)	Pool elevation in advance of floods (5)	Minimum elevation for navigation (6)	Spillway crest elevation (7)	Spillway length, in feet (8)
Knoxville	647.7	8 990	....	....	....	....	....
Coulter Shoals	603.5	9 600	815.0	806.0	804.5	775.0	400
Watts Bar	529.9	17 400	746.0	740.0	740.0	721.0	760
Chickamauga	471.0	20 800	685.0	675.0	673.5	645.0	720
Hales Bar	431.1	22 000	....	....	....	....	....
Guntersville	349.0	24 300	595.0	591.0	590.0	555.0	720
Wheeler	274.9	29 600	558.0	550.0	546.0	541.0	2 400
Wilson	259.4	30 800	....	....	....	....	....
Pickwick Landing	206.7	32 900	418.0	408.0	408.0	378.0	880
Gilbertsville	22.5	40 200	370.0	352.0	352.0	330.0	840

RATING CURVES

Natural Condition at Dam Sites.—Most of the gaging stations on the Tennessee River and its main tributaries from Knoxville to the mouth were used in some way in this work. Where discharge measurements have been made, rating curves based on the latest data have been used. At some gaging stations where river-stage records are available but where no discharge measurements have been made, and at dam sites, rating curves have been constructed by using stage-relation curves between the gaging station or dam site in question and the nearest gaging stations which have been rated. In some cases, particularly at dam sites, it was necessary to construct or extend the rating curves by means of back-water curve computations. Only the rating curves which have been made from actual flow measurements can be taken without reservation. All rating curves constructed by other methods and all extensions of rating curves must be accepted subject to revision when better data are available.

At all routing stations except the Pickwick Landing Dam site and the Gilbertsville Dam site, the rating curves predicated a constant relationship between stage and discharge. At the Pickwick Landing Dam site a "changing-stage" rating curve was constructed to take account of slope and back-water effects. It was based on a normal curve and on two other curves—one for falling stages and one for rising stages—giving the percentage correction for each foot change of stage from the preceding day for any elevation. The normal rating curve was derived from back-water computations and other data, and the correction curves were determined by trial routings of several floods. The changing stage rating curve, then, consisted of a family of curves plotted with elevation against discharge and with the change in stage from the preceding day as the parameter, or third variable.

The method used in obtaining the Gilbertsville rating curve is similar to that developed by the U. S. Geological Survey, in the Missouri (St. Louis) District

Office, in the construction of a rating curve for the Ohio River, at Paducah. The relation between percentage submergence and a submergence factor was determined from the data available. For any percentage submergence and stage at Gilbertsville the actual discharge was obtained by applying the submergence factor to the actual stage to determine an effective stage. Then, with this effective stage, the corresponding discharge was determined from a base rating curve. For convenience, the rating curve was constructed, from the foregoing information, as a family of curves with discharge at Gilbertsville as abscissa, elevation at Paducah as ordinate, and elevation at Gilbertsville as the third variable.

Tail-Water and Head-Water Curves.—The tail-water rating curves at the dams are the same as the rating curves in the natural condition modified, in most cases, by the back-water from the nearest reservoir down stream.

The spillway discharge curves were based on the usual weir formula, $Q = C\,L\,H^{3/2}$, and were computed for unregulated conditions only; that is, all gates open. Values of the coefficient, C, were taken the same as those used by the U. S. Army Engineers.[5] When the spillway was submerged, the coefficient, C', as obtained from the results of the United States Deep Waterways experiments[6] on submerged-weir models, was substituted in the weir formula for the coefficient, C. The resulting spillway discharge curves were single-line curves, except for the Pickwick Landing and Gilbertsville Dams. At these two dams it was necessary to construct a family of curves, each member being based on a fixed tail-water elevation.

Inflow

Inflow into each reach was divided into two parts: (1) The discharge from the reach immediately above; and (2) the local and tributary inflow, which consisted of the measured flow at U. S. Geological Survey gaging stations on the larger tributaries, together with an estimated flow from the unmeasured area. The sum of the discharge from the upper reach and the local and tributary flow is the total inflow, I, which is used in solving the storage equation. If the distance of a tributary station from the main stream is great, the flows passing the gaging station on any day were assumed to enter the main river storage one or two days later. This time lag, to the nearest day, was determined from a study of the time of travel of one particular flood.

The first step was to obtain the relation between the drainage area and the volume in the flood hydrograph at the various dam sites. This was done by plotting the flood volumes at the regular U. S. Geological Survey gaging stations against the drainage areas. The drainage area at the dam site then determined the volume of the flood at that point. The total volume to come from rainfall on the unmeasured area was determined from the difference between the volumes of the flood hydrograph at the upper and lower ends of each reach. Table 2 shows the total volume for the 1926–1927 flood at each location, the volume from the measured tributaries in each reach, and the volume from the unmeasured area. (In Table 2 the total volume in 1 000 sec-ft days is not

[5] H. R. Doc. No. 328, 71st Cong., 2d Session, Appendix 1, Chart CIV.

[6] "Weir Experiments, Coefficients, and Formulas," by Robert E. Horton, M. Am. Soc. C. E., U. S. Geological Survey *Water Supply and Irrigation Paper No. 200.*

necessarily the exact figure as taken from the total volume-drainage area relation. The total volume for the local areas is obtained from this relation, as previously stated, but the total volume shown to any point is the sum of the

TABLE 2.—Total, Tributary, and Local Volume of 1926–1927 Flood

Location	Drainage area, in square miles	Total volume, 1 000 second-foot days (29 days)	Measured Tributaries		Local Areas, Unmeasured					
			Drainage area, in square miles	Volume, 1 000 second-foot days (29 days)	Drainage area, in square miles	Volume, 1 000 second-foot days (29 days)	Base flow, in cubic feet per second	Surface run-off, 1 000 second-foot days (30 days)	Rainfall, in inches	Surface run-off, percentage
Knoxville, Tenn.	8 990	886	..	..	..	..	..	..	..	..
Knoxville to Coulter Shoals	610	83	235	30	375	53	400	44	6.92	63.1
Coulter Shoals, Tenn.	9 600	968	..	..	..	..	..	..	..	..
Coulter Shoals to Watts Bar	7 800	1 345	5 662	1 009	2 138	336	2 800	275	8.25	58.0
Watts Bar, Tenn.	17 400	2 297	..	..	..	..	..	..	..	..
Watts Bar to Chickamauga	3 400	544	2 300	299	1 100	245	2 030	189	7.65	83.4
Chickamauga, Tenn.	20 800	2 838	..	..	..	..	..	..	..	..
Chickamauga to Hales Bar	1 200	259	..	..	1 200	259	..	..	..	..
Hales Bar, Tenn.	22 000	3 100	..	..	..	..	..	..	..	..
Hales Bar to Guntersville	2 300	445	389	85	1 911	360	3 000*	273	6.87	77.4
Guntersville, Ala.	24 300	3 566	..	..	..	..	..	..	..	..
Guntersville to Wheeler	5 300	1 097	1 700	388	3 600	709	5 412	556	8.92	64.4
Wheeler, Ala.	29 600	4 678	..	..	..	..	..	..	..	..
Wheeler to Wilson	1 200	237	355	71	845	167	930*	141	9.36	66.5
Wilson, Ala.	30 800	4 878	..	..	..	..	..	..	..	..
Wilson to Pickwick	2 100	474	621	141	1 479	333	1 850*	284	9.36	76.4
Pickwick, Tenn.	32 900	5 372	..	..	..	..	..	..	..	..
Pickwick to Gilbertsville	7 300	1 588	3 049	684	4 251	904	6 400	742	8.40	78.1
Gilbertsville, Ky.	40 200	7 108	..	..	..	..	..	..	..	..

* Average for the period.

discharges obtained by routing. Unless the storage stage at the end of the routing period is the same as that at the beginning and unless the outflow is the same as the inflow at the beginning and end of the routing period, the total volume of outflow cannot equal the total volume of inflow. For instance, the sum of the Watts Bar total volume of 2 297 000 and that for the area between Watts Bar and Chickamauga of 544 000 is 2 841 000 and is the volume of inflow into the Chickamauga Reach, but the volume of discharge passing Chickamauga Dam site is 2 838 000 as obtained by routing.) The distribution of the unmeasured local inflow was obtained from rainfall and a distribution graph, the latter being determined from distribution graphs of streams of about the same drainage area and shape as those in each reach herein considered. The procedure for transforming rainfall into run-off by means of a unit graph was developed by Leroy K. Sherman,[7] M. Am. Soc. C. E., and enlarged by Merrill Bernard,[8] M. Am. Soc. C. E.

In every reach the unmeasured area was relatively a small part of the total drainage area above that point, the maximum being 12.3%, so that the dis-

[7] "Stream Flow from Rainfall by the Unit Graph Method," by Leroy K. Sherman, *Engineering News-Record*, Vol. 108, 1932, pp. 501–505.

[8] "An Approach to Determinate Stream Flow," by Merrill Bernard, *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 347.

tribution graph could be somewhat in error without materially affecting the discharge.

The same local and tributary flows were used in the case with dams as in the natural condition, modified where necessary, of course, in accordance with the assumed operation of tributary storage dams.

The local and tributary inflow can be obtained by working backward from known daily discharges and storage on the main stream, but in every trial of this method the results were found to fluctuate. This fluctuation is due to placing all the error in the local and tributary flow, since a relatively small error in the main river discharge would be a large error in the local and tributary inflow. Then, if on one day, the computed local and tributary inflow is too low, the next day it will be correspondingly higher and the next day that much lower, etc., until negative quantities of local and tributary inflow result. However, if the fluctuating values obtained by working backward are smoothed graphically, reasonable local and tributary flows are obtained. In the example (see heading, "Example of Routing—Chickamauga Reach") given in this paper this method was not used.

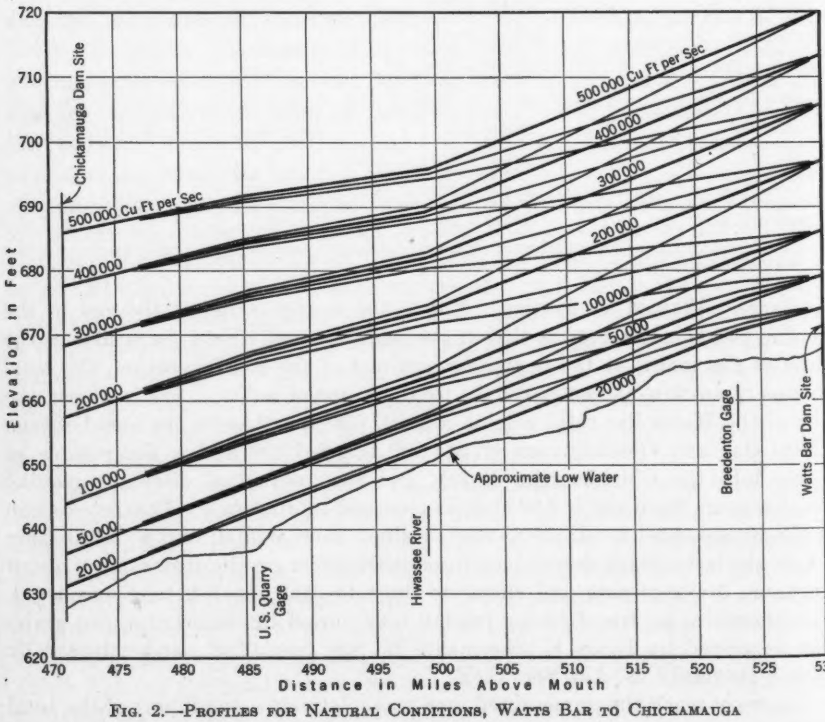

FIG. 2.—PROFILES FOR NATURAL CONDITIONS, WATTS BAR TO CHICKAMAUGA

STORAGE

Each reach was considered as a reservoir, but instead of assuming the reservoir as level, storage was computed from the volume under a sloped profile

based on rating curves and back-water studies. In the natural condition, when there was a constant relation between elevation and discharge, profiles were drawn for a series of steady flows, the highest one being at least as large as the largest flood to be routed. In order to obtain a series of storage curves for use in routing that would cover the entire range of conditions, profiles of non-steady flow were drawn between the steady flow profiles. A typical set of these curves is shown in Fig. 2. The manner in which the flow was varied from the upper end of the reach to the lower end differed for each reach. In a reach where there was no large tributary, the flow was varied directly with the length of the reach; for example, with 200 000 cu ft per sec flowing at the upper end, and 300 000 cu ft per sec flowing at the lower end, the flow at the mid-point of the reach was taken to be 250 000 cu ft per sec. The elevation at the mid-point was determined by that flow, applied to a rating curve drawn from the calculated back-water curves. In reaches where a relatively large tributary entered, the flow was varied on the basis of drainage area; for example, if the drainage area from the upper end of the reach to just below the mouth of the tributary is three-fourths of the total drainage area in the reach, then the flow at that point is taken as 275 000 cu ft per sec for the same flow conditions as given previously.

To represent fully the condition after the dams are built, the storage curves for each reach must be in such a form as to satisfy any flow conditions in that reach for any fixed head-water elevation at the dam at its lower end. This is necessary because the discharge from any dam may possibly be regulated to maintain any elevation required. To provide the necessary data, three sets of profiles were drawn—one set for an unregulated condition (that is, all gates open), and the other two sets for regulated conditions of fixed elevations at the dam. The fixed elevations usually selected were the maximum flood surcharge elevation and the minimum navigation elevation. Typical curves are shown in Figs. 3 and 4, and their use is explained under the routing procedure described in the example (see heading, "Example of Routing—Chickamauga Reach").

Storage under each of the profiles was obtained by dividing the reservoirs into smaller reaches varying from 2 miles in length on the upper river to 10 miles in length on the lower river. A volume curve was obtained for each of these smaller reaches, either by planimetering a contour map or by taking cross-sections and assuming them to be average for a certain distance. Where the latter method was used, the final individual volume curves were adjusted so that the total volume from them for a horizontal pool would agree with a level-reservoir volume curve which had been obtained by planimetering the best available maps. Low-water elevation was taken as the point of zero volume for all profiles. The total storage under the profile was obtained by summing up the volumes corresponding to the elevations taken from the profiles at the mid-points of the individual reaches. When outflow-plus-storage curves were required (as in the natural condition), the discharge at the lower end of the reach, for the corresponding profile, was added to the storage. This sum was plotted as abscissa, with discharge as ordinate, and with the discharge at the

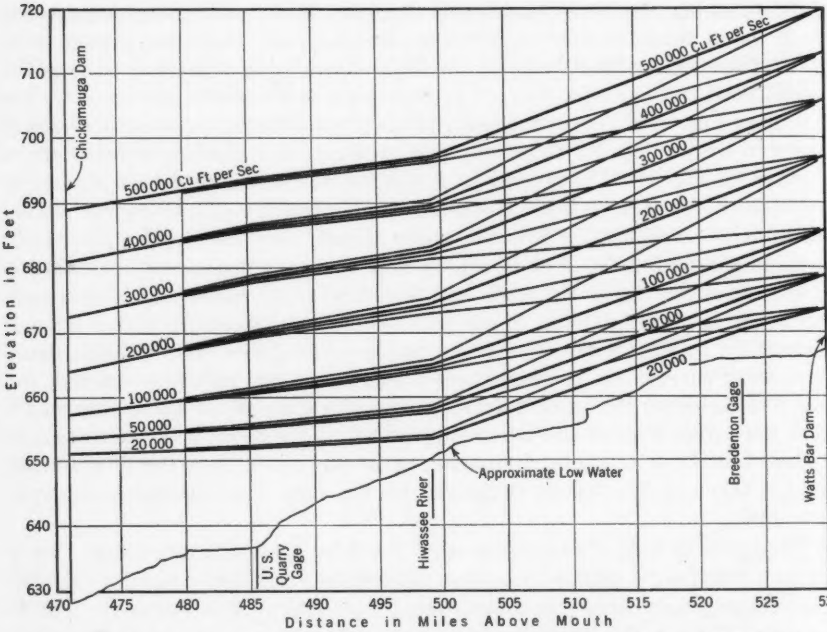

FIG. 3.—PROFILES FOR UNREGULATED CONDITION, WITH DAM, WATTS BAR TO CHICKAMAUGA

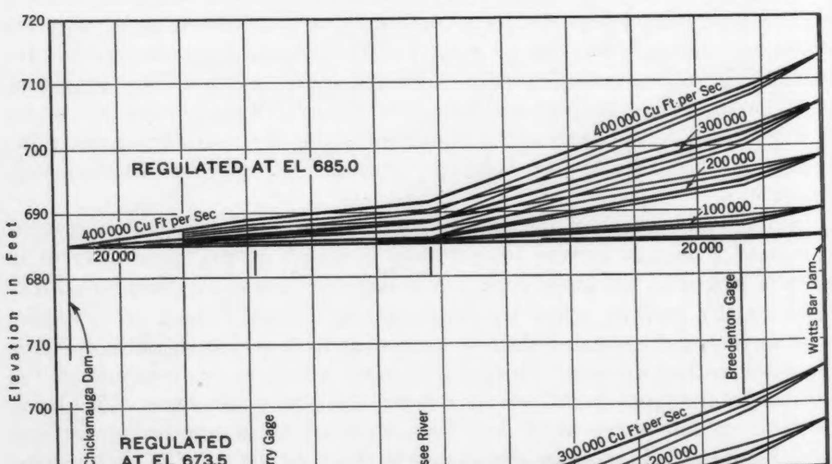

FIG. 4.—PROFILES FOR REGULATED CONDITION, WITH DAM, WATTS BAR TO CHICKAMAUGA

upper end of the reach as the parameter, thus forming a family of curves (see Fig. 5). For the conditions after the dams are built, storage alone is plotted as abscissa (Fig. 6). For each of the five upper reaches (Coulter Shoals, Watts Bar, Chickamauga, Guntersville, and Wheeler) there is one set of curves

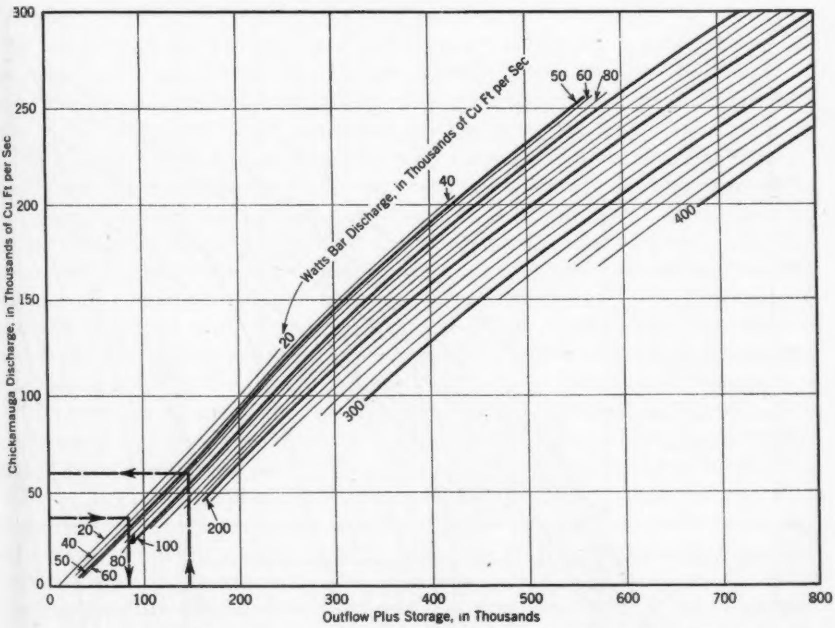

FIG. 5.—OUTFLOW-PLUS-STORAGE CURVES FOR NATURAL CONDITION, CHICKAMAUGA REACH

of outflow plus storage, for use in the natural condition, and three sets of curves of storage, for use after the dams are built.

For the two lower river reaches (Pickwick Landing and Gilbertsville) a different procedure was used for routing in the natural condition, because of the effect of slope and back-water on the stage-discharge relation. In each of these reaches three sets of outflow-plus-storage curves were constructed, based on three different rating curves for the respective dam sites, in order to allow for different discharges that can occur for any given elevation at the lower end of the reach. At Pickwick Landing, the rating curves selected were the normal curve, the 10-ft rise-in-stage curve, and the 4-ft fall-in-stage curve. The resulting sets of outflow-plus-storage curves, were plotted against the Pickwick elevation, with discharge at Wilson Dam as the third variable. In the routing procedure the true elevation, discharge, and storage are interpolated from the three sets; this process is explained under routing procedure (see heading, "Routing"). At Gilbertsville, the rating curves are based on the elevation of the Ohio River at Paducah, which is assumed to be known in all Gilbertsville routings. The rating curves used in this case were for three different percentage submergences of the Gilbertsville gage, and the resulting outflow-plus-storage

curves were plotted against the Paducah elevation, with the Pickwick elevation as the third variable.

For routing after the dams are built, the storage curves for the two lower reaches were drawn in the same manner as those for the upper reaches. At all

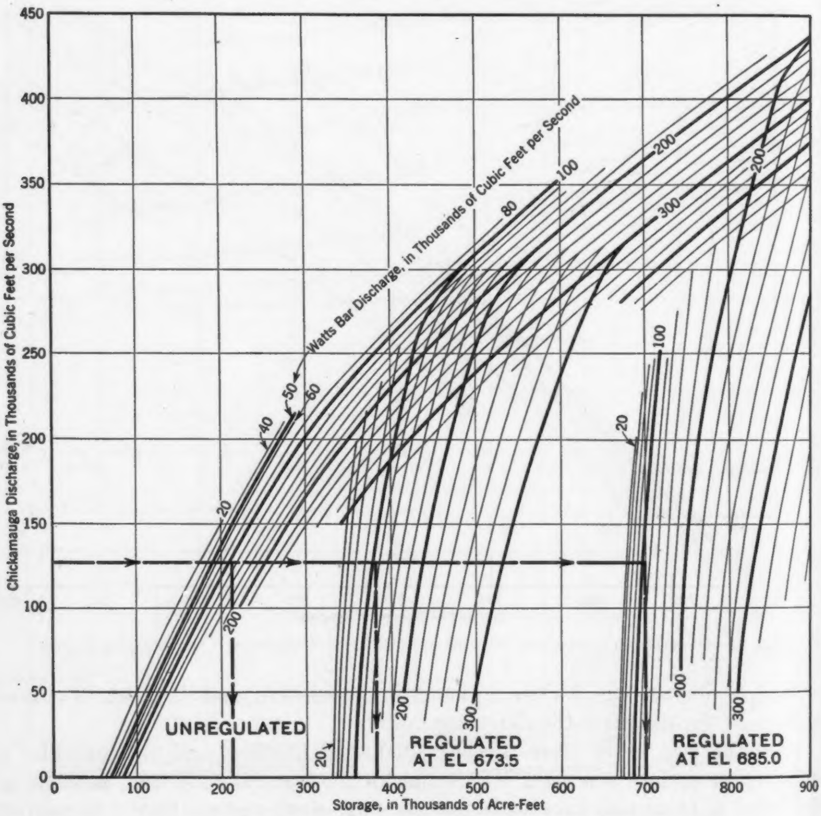

FIG. 6.—STORAGE CURVES FOR CHICKAMAUGA DAM

dams, when an unregulated condition is assumed, the routing procedure is exactly the same as in the natural condition, except that the outflow-plus-storage curves are based on the rating curve for unregulated head-water elevation.

Other methods of obtaining storage were considered in the course of this study. Throughout the period covered by a flood hydrograph it is possible to determine the storage change on each day if the discharge at the upper and lower ends of the reach and the local and tributary inflow are known. Then, knowing the initial storage, the total storage at the end of each time period can be determined. This method has been described by Harold A. Thomas,[9]

[9] "The Hydraulics of Flood Movements in Rivers," by Harold A. Thomas, *Engineering Bulletin*, Carnegie Inst. of Technology, p. 65.

M. Am. Soc. C. E. The objections to its use are that the hydrograph at each end of the reach must be known fairly accurately (which was not the case in this study) and that storage curves obtained by this method are not easily adapted to the condition with the dams built. It is also necessary to have a different storage curve for each flood or an average curve based on a number of floods.

If the profiles and the storage under them are correct, it is not necessary to introduce any time element into the routing procedure, provided the length of each reach is such that the true storage within it at any time can be determined by the end elevations. The profiles show the storage at a particular instant, and the storage conditions in the entire river are considered at the same instant. It is believed that if the length of the reach is such that the time of travel is approximately equal to the time period used in the routing, the use of the two end elevations in fixing the profile of the water surface, and, consequently, the storage, will result in reasonable accuracy. The Gilbertsville Reach was the only one in which a correction was necessary to allow for an extreme length, which in this case was 184 miles. Since the time of travel in this reach is several days, a flood wave passing through the reach does not agree with the assumptions used in the shorter reaches as to surface profiles and storage.

One purpose of routing in the natural condition to check an actual hydrograph was to learn whether the routing would give an accurate indication of the time of occurrence of the peak. It was found that it did; there was no marked difference between the observed and routed time. It is an accepted fact that an increase in flow at the upper end of a reach is transmitted to the lower end more rapidly through a reservoir than along the river in its natural condition. That the routing procedure for the condition with dams automatically takes this into account will be clear from inspection of the profiles for the natural and regulated conditions for a typical reach (Figs. 2 and 4) and the corresponding outflow-plus-storage curves or storage curves (Figs. 5 and 6). For example, assume that a steady flow of 100 000 cu ft per sec throughout the reach is increased to 200 000 cu ft per sec at the upper end. The increment of storage that must be filled before an outflow of 200 000 cu ft per sec can occur at the lower end of the reach is: (1) For the natural condition, 174 000 acre-ft; (2) for the dams in place, with regulation at Elevation 673.5, 100 000 acre-ft; and (3) for regulation at Elevation 685.0, 70 000 acre-ft. The routing method thus provides for the faster passage of water through the reservoirs, and its accuracy is limited only by that of the profiles and storage.

Example of Routing—Chickamauga Reach

The Watts Bar-to-Chickamauga Reach is now discussed in detail as an example of the routing procedure.

Chickamauga Inflow.—Inflow into the upper end of this reach was made the same as the routed discharge from the Watts Bar Reach immediately above. (The Watts Bar routing is not shown in this example.)

The increase in drainage area between the upper and lower ends of the Chickamauga Reach, as shown in Table 2, is 3 400 sq miles. The run-off from 2 300 sq miles was measured on the Hiwassee River at Charleston, Tenn., leav-

ing the run-off from 1 100 sq miles to be estimated from an average rainfall of 7.65 in. in 9 days. This average rainfall was obtained from six stations corrected to a calendar-day basis. The unmeasured drainage area is made up of many small streams running directly into the main river and, for this reason, a distribution graph was used which gave a relatively quick run-off. The percentages used were 50, 30, 12, 5, 2, and 1, which means that 50% of the run-off from a day's rain arrived in the reach on that day, 30% on the next day, etc. The total volume of run-off from the unmeasured area was obtained as follows: (1) From the total volume-drainage area relation for a 30-day period, the total flood volume at Watts Bar was 2 346 000 sec-ft days, and that at Chickamauga was 2 899 000 sec-ft days; (2) the difference (553 000 sec-ft days) of these two volumes was the total flood run-off from the area between Watts Bar and Coulter Shoals; (3) the total flood volume of the Hiwassee River, at Charleston, Tenn., was 303 000 sec-ft days, and this volume was subtracted from 553 000 to give 250 000 sec-ft days, the total flood volume from the unmeasured area; (4) the base flow of 2 030 cu ft per sec per day or 61 000 sec-ft days was subtracted from 250 000 to give 189 000 sec-ft days, the total volume from rainfall of 7.65 in. on the unmeasured area. Since there was some rainfall on the day preceding the beginning of the routing period, the quantities given for a 30-day period are somewhat greater than those shown in Table 2 for a 29-day period. The percentage of run-off was assumed to be constant for the entire storm.

The sum of the flow of the Hiwassee River, at Charleston, and the estimated flow from the unmeasured area was added, in the routing, to the inflow at the upper end of the reach to give the total inflow, I, used in the storage equation.

When the Hiwassee Dam was assumed to be in operation, the flow at that point was interpolated from the records of the Hiwassee River at Reliance, Tenn., and Murphy, N. C., and the quantity stored at Hiwassee Dam was subtracted from the Chickamauga local and tributary flow one day later than it occurred at the Hiwassee Dam site.

Chickamauga Storage.—Natural storage in the Chickamauga Reach was obtained as follows: (1) A series of back-water curves for various non-steady flows were computed, based on values of n in Kutter's formula as derived from a study of several known flood profiles and a Chickamauga rating curve determined from gage relations with the Chattanooga curve; (2) from the calculated curves, rating curves were drawn at all required points in the reach; (3) from these rating curves the elevations at the desired flows were taken and flow profiles were constructed (Fig. 2); and (4) storage under each profile was obtained by summing the 2-mile reach volumes which had been planimetered from topographic maps made in 1924 by the U. S. Corps of Engineers. To the storage was added the outflow at Chickamauga, and this sum was plotted against discharge at Chickamauga with the discharge at Watts Bar as the third variable (Fig. 5). In explanation of Step (3), it should be noted that, the non-steady flow was assumed to vary with the drainage area; for example, the drainage area between Chickamauga and the mouth of the Hiwassee River is 380 sq miles; the drainage area of the Hiwassee River is 2 660 sq miles; and the total drainage area between Watts Bar and Chickamauga is 3 400 sq miles. Then, if the flow is 300 000 cu ft per sec at Chickamauga and 100 000 cu ft per sec at Watts Bar,

it is assumed to decrease uniformly from 300 000 at Chickamauga to 300 000 $-\frac{380}{3\,400}$ (300 000 − 100 000) at the mouth of the Hiwassee River. At that point it is further decreased by $\frac{2\,660}{3\,400}$ (300 000 − 100 000), and from there it is decreased uniformly to 100 000 at Watts Bar.

Three sets of profiles for the conditions after the dam is built (Figs. 3 and 4) were constructed in a similar manner. However, for the cases shown in Fig. 4 (which apply when the outflow from the dam is being regulated) the non-steady flow profiles were varied with the distance instead of the drainage area, as it was believed that with regulation the character and shape of a flood is so changed that the tributary does not necessarily contribute inflow in the same proportion as in the natural condition. Storage under the profiles was obtained from the same 2-mile reach volume curves that were used for the natural condition. In this case, however, storage was plotted against discharge at Chickamauga with discharge at Watts Bar as the third variable (Fig. 6).

Chickamauga Routing.—A sample of routing in the natural condition (the latter part of the flood has been omitted) from Watts Bar to Chickamauga is given in Table 3(a).

TABLE 3.—EXAMPLES OF ROUTING, CHICKAMAUGA REACH

(Flows and Storages Are in Units of 1 000 Cubic Feet per Second and 1 000 Acre-Feet, Respectively.)

Item	Values for the Following Calendar Days in December, 1926										
	21	22	23	24	25	26	27	28	29	30	31
(a) NATURAL CONDITIONS											
Watts Bar discharge	29	76	95	138	187	188	200	209	189	179	146
Local and tributary inflow	16	16	24	40	54	52	49	52	47	35	24
Total inflow, I_1	45	92	119	178	241	240	249	261	236	214	170
Total inflow, I_2	92	119	178	241	240	249	261	236	214	170	129
Storage, S_1	48	88	135	187	278	345	370	396	394	355	291
$I_1 + I_2 + S_1$	185	299	432	606	759	834	880	893	844	739	590
Chickamauga discharge, O_1	37	60	104	141	187	227	237	247	252	237	211
$O_2 + S_2$	148	239	328	465	572	607	643	646	592	502	379
Elevation at dam site	635.2	639.1	645.4	650.0	655.8	660.5	661.7	662.9	663.4	661.7	658.7
(b) WITH DAMS											
Watts Bar regulated discharge*	70	76	91	112	112	112	112	112	112	112	110
Local and tributary inflow†	14	14	21	37	50	40	35	45	37	21	18
I_1	84	90	112	149	162	152	147	157	149	133	128
I_2	90	112	149	162	152	147	157	149	133	123	....
S_1	387	373	369	395	452	512	557	607	659	687	694
Sum	561	575	630	706	766	811	861	913	941	948	....
Regulated Chickamauga discharge	90	98	108	127	127	127	127	127	127	127	127
Head-water elevation	675.0	674.1	673.5	674.1	676.7	679.1	680.7	682.3	683.8	684.6	684.9
Tail-water elevation	643.4	644.5	646.0	648.3	648.3	648.3	648.3	648.3	648.3	648.3	648.3

* Modified from the natural case, Table 3(a), due to storing in Norris Reservoir and regulation at Coulter Shoals and Watts Bar Reservoirs.

† Modified from the natural case, Table 3(a), due to storing in Hiwassee Reservoir.

The Watts Bar discharges or inflows at the upper end of the reach are those obtained by routing from up stream. The Chickamauga discharge, O_1, is estimated for the first day to be 37 000 cu ft per sec. From Fig. 5, using the Watts Bar and Chickamauga flows for December 21, 1926, a value of 85 000 is obtained for $O + S$, and subtracting O_1, $S_1 = 48\,000$ acre-ft. (This interchange of units of flow and storage has been explained under the heading, "General Procedure.") The total inflow, I_1, for December 22, 1926, is brought over as I_2 of December 21, and the sum of 185 000 is obtained, from which the Chickamauga O_1 of 37 000 is subtracted, giving an $O_2 + S_2$ of 148 000 for December 21, which becomes the $O_1 + S_1$ for December 22. This value of outflow plus storage, the Watts Bar discharge of 76 000 for December 22, and the $O + S$ curves determine the Chickamauga outflow, O_1, of 60 000, and the storage, S_1, of 88 000 (that is, 148 000 − 60 000) for December 22. The procedure is then repeated for as many periods as desired. Table 3(a) shows a peak Chickamauga discharge of 252 000 sec-ft on December 29. The elevations are obtained from the Chickamauga tail-water curve (Fig. 7). The routed

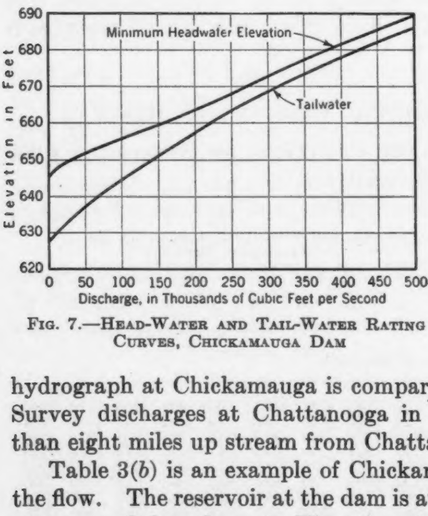

Fig. 7.—Head-Water and Tail-Water Rating Curves, Chickamauga Dam

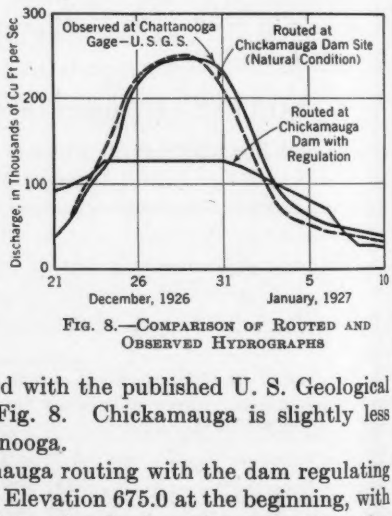

Fig. 8.—Comparison of Routed and Observed Hydrographs

hydrograph at Chickamauga is compared with the published U. S. Geological Survey discharges at Chattanooga in Fig. 8. Chickamauga is slightly less than eight miles up stream from Chattanooga.

Table 3(b) is an example of Chickamauga routing with the dam regulating the flow. The reservoir at the dam is at Elevation 675.0 at the beginning, with an assumed drawdown to Elevation 673.5 and a flood surcharge to Elevation 685.0 (Table 1).

The constant outflow to which it is possible to regulate a particular flood without violating the assumed elevations is estimated and then checked by routing, and if a greater or less reduction is indicated, the routing is revised.

The outflows were regulated as desired and in the given example made such, from December 21 to December 23, 1926, that the reservoir elevation at the dam would be drawn down to the minimum allowable and held there as long as possible without exceeding the flow to which it was possible to reduce the main peak of the flood. The constant outflow of 127 000 cu ft per sec was made such that the reservoir elevation at the dam would not exceed Elevation 685.0 at any time.

The principal problem of routing with regulated operation of the dams is to determine the correct head-water elevation from day to day. As in all the routing, the storage on the first day must be determined from the river conditions. On succeeding days the storage is then determined from the regulated Watts Bar inflows, the local and tributary inflows, and the regulated Chickamauga outflows.

For a particular day, such as December 24, the three sets of storage curves in Fig. 6 are entered with the Watts Bar inflow of 112 000 cu ft per sec and the Chickamauga discharge of 127 000 cu ft per sec. A storage of 214 000 acre-ft is obtained from the curves for the unregulated condition, 382 000 acre-ft from the set for regulation at Elevation 673.5, and 700 000 acre-ft from the set for regulation at Elevation 685.0. The elevation corresponding to the storage for the unregulated condition is found to be 658.0 from the head-water curve of Fig. 7 for the Chickamauga flow of 127 000 cu ft per sec. The three storage

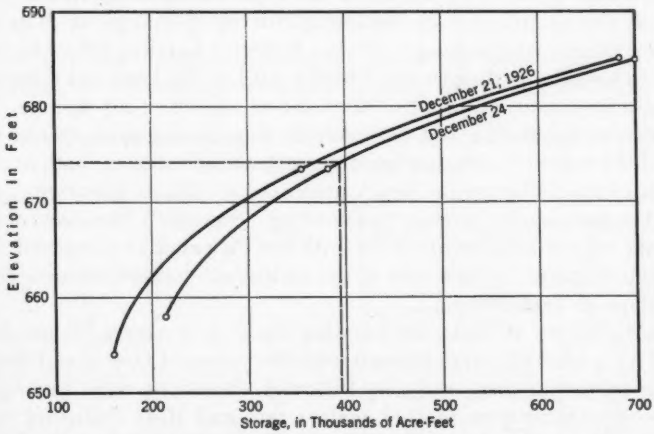

FIG. 9.—RELATION BETWEEN ELEVATION AND STORAGE ON TYPICAL DAYS, CHICKAMAUGA REACH, WITH DAMS

values are plotted against their respective elevations (Fig. 9) and a curve is drawn through the points. Then, entering Fig. 9 with the actual storage for December 24 (395 000 acre-ft), the corresponding elevation of 674.1 is determined for that date. The process is repeated for each day.

The tail-water elevations are determined by applying the regulated Chickamauga discharges to the tail-water rating curve of Fig. 7.

SPECIAL PROBLEMS

In routing from Knoxville to Gilbertsville a number of special problems were encountered. The Knoxville hydrograph must be known to route through the Knoxville-Coulter Shoals Reach, and the regular procedure is then followed in routing through Watts Bar and Chickamauga.

No attempt was made to route through the Hales Bar Pool. In the natural routing the Hales Bar flows were obtained by applying gage heights to the latest

rating curve and were used in routing through the Guntersville Reach. All the Guntersville storage curves were prepared on this basis.

The natural routing thus required no local inflow for the area between Chickamauga and Hales Bar. However, with the dams operating a local inflow was necessary, and it was obtained by plotting the Chickamauga and Hales Bar hydrographs on separate sheets to the same scale. The Chickamauga plotting was placed on the other and lagged in time, a distance approximately equal to the time of travel between the two points. The difference between the discharges on each day, using the date on the Hales Bar hydrograph, was taken as the local inflow on that day and was assumed to represent any storage effect of the Hales Bar Pool. These values were added to the Chickamauga regulated outflows to obtain the inflows into the upper end of the Guntersville Reach. The Guntersville and Wheeler Reaches were routed in the regular manner.

The floods were not routed through the Wilson Reservoir. For the natural condition, inflow into the upper end of the Pickwick Landing Reach was made the same as the published U. S. Geological Survey discharges at Florence, Ala. For the condition with the dams built, the Pickwick Landing inflow at the upper end was obtained by adding to the Wheeler outflow the local and tributary flow into Wilson Reservoir, allowing no time lag or reduction for storage.

The Pickwick Landing and Gilbertsville Reaches required special methods because of the types of rating curves necessary at each station; and, in addition, the length of the Gilbertsville Reach presented a difficult situation. As mentioned in the discussion of storage (see heading, "Storage"), three sets of outflow-plus-storage curves were prepared for both the Pickwick Landing and Gilbertsville natural routings, to take care of the additional variable introduced by the effect of slope or back-water.

For rising stages at Pickwick Landing the $O + S$ curves for no change of stage and for a 10-ft rise were entered with the values of $O + S$ and the Wilson flow for a particular day, and two Pickwick elevations were thus obtained. These two elevations were plotted against zero and 10-ft change of stage, respectively, and connected by a straight line. Then, by trial, a change in stage was determined from this plotting such that when added to the previous day's elevation, the new elevation was that given on the curve for the change in stage used. Falling stages were handled similarly except that the discharge plus storage curves for zero change of stage and those for a 4-ft fall were used. Having found the elevation and change of stage, the discharge for that day was known from the rating curve.

At Gilbertsville, the treatment was similar to that at Pickwick Landing, except that the additional variable taken care of by the three sets of outflow-plus-storage curves was percentage submergence. In addition, all three sets were used in the supplementary plotting to obtain the true Gilbertsville elevation.

An additional variation was made in the natural Gilbertsville routing to take care of the length of the reach. For use in entering the outflow-plus-storage curves, the Pickwick Landing elevation was adjusted. The time of travel was assumed to be about three days, and the effect of a given day's change in stage at Pickwick Landing was spread over three days.

The routing with the dams built at Pickwick Landing and Gilbertsville was slightly different from the example for Chickamauga. The difference at Pickwick Landing was in obtaining the reservoir elevation against which to plot the storage figure obtained from the storage curves for the unregulated condition. The Pickwick tail-water elevations were obtained by trial and error from the changing-stage rating curve before the head-water elevation could be obtained from the special head-water curves previously described.

For routing in Gilbertsville, the Pickwick Landing tail-water elevations were adjusted in the same manner as in the natural routing. Here, again, the tail-water elevation was obtained before the head-water elevation could be found. The Paducah elevation, corrected for the regulation of the Tennessee River, the natural rating curve, and the regulated outflow, fixed the tail-water elevation. Knowing the tail-water elevation and the discharge, the head-water elevation was fixed for use with the set of unregulated storage curves.

Conclusions

Table 4 gives the results of routing for two of the cases studied. The dams were operated in accordance with the limiting elevations given in Table 1.

TABLE 4.—Peak Discharge in Thousands of Cubic Feet per Second at Main-River Dam Sites

Location	Flood of 1926–1927		1926–1927 Flood Plus 50 Per Cent	
	River in natural condition	Dams in place*	River in natural condition	Dams in place*
Coulter Shoals	85	64	125	103
Watts Bar	209	112	307	180
Chickamauga	252	127	358	216
Guntersville	286	162	407	276
Wheeler	352	229	504	418
Pickwick Landing	375	270	595	500
Gilbertsville	367	344	545	510
Columbus, Ky., on the Mississippi River	1 320	1 180	1 580	1 448

* Norris, Hiwassee, and all main-river dams.

Elevations at the beginning of the flood period were assumed as those in Column (5) of that table. A drawdown was used where allowed by minimum navigation limits (Column (6)). The flood surcharge elevations used are given in Column (4). It must be kept in mind that this range of operation of the dams is tentative and subject to change. The flood, 50% greater than that of 1926–1927, was built up by increasing the Knoxville hydrograph by 50% and then routing down stream with the local and tributary inflow of each reach increased by 50 per cent.

It is believed that the procedure developed gives results as accurate as can be obtained with the data available. The storage curves and profiles as compiled in this paper are not suitable for low flows or sharp-peaked floods of short duration, but with some modification they could be made applicable.

For floods as great as any that have occurred and for which fairly accurate ratings are available at the various stations, the method seems satisfactory in studying past floods or in forecasting flows for desirable operation of dams as future floods occur.

In forecasting, the beginning point at Knoxville and the local and tributary inflows must be predicted as far into the future as it is desired to forecast the main river. This is best done by consideration of rainfall and extension of tributary records as they become available.

ACKNOWLEDGMENTS

The writers wish to express their appreciation to J. H. Kimball, M. Am. Soc. C. E., Head of the Flood Control Section of the TVA, under whose direction the studies described in this paper were made; and to B. J. Buehler, Jun. Am. Soc. C. E., for reading and criticizing the manuscript; and to other members of the Flood Control Section for preparation of some of the material used in the study.

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PAPERS

DESIGN OF PILE FOUNDATIONS

BY C. P. VETTER,[1] M. AM. SOC. C. E.

SYNOPSIS

It is intended, in this paper, to present rational methods for the determination of stresses in the individual piles of a pile foundation which supports a rigid structure such as a bridge pier or a hydraulic gate pier.

Athough it may be formulated in different ways, the purpose of the piles is obviously to minimize the settlement of the pier when it is acted upon by certain exterior forces. For the purpose of this investigation, the weight of the pier and other mass forces are considered exterior forces.

It is evident that the function of the piles is merely one of transfer. The statement is frequently made that the piles "carry" the structure; it would be more logical to state that the piles transfer the load from the structure to such strata of the foundation as are capable of carrying the load with a resultant settlement less than that prescribed for the structure. It is the subsoil that "carries" the load, and consolidation of soft material below the piles will inevitably contribute its share to the settlement of the structure whether or not piles are used.

If all external loads are vertical and the piles are also vertical, the piles transfer the load to deep layers in the subsoil. These layers may be more solid than the surface layers and may be so composed that they will support the load without material settlement. However, they may also be of essentially the same composition as the surface layers, but be more consolidated due to the superimposed load of the upper layers, so that the increased load of the structure may not cause objectionable settlements. Thus, the function of the piles is to transfer the load to the subsoil in such a manner that the compressibility of the upper layers of the foundation material affects the settlement of the structure only to a minor extent, if at all. Again, it may happen that the upper layers are of such composition that they will consolidate only slightly under load, whereas the lower layers are softer. Under such circumstances, there is obviously no

NOTE.—Written comments are invited for immediate publication; to ensure publication the last discussion should be submitted by **June 15, 1938.**

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reason for using piles; in fact, the driving of piles may so disturb the upper layers that their original compactness is destroyed with a resultant greater settlement of the structure than would have occurred had no piles been used.

If the external loads are horizontal and the piles are vertical, conditions are entirely different. There is, now, no transfer of load to deeper foundation layers, except such as is caused by the stiffness of the piles, which, especially in the case of wooden piles, is relatively small. The horizontal load is transferred mainly to the upper foundation layers by means of direct pressures between the sides of the piles and the soil. Although the upper layers may be solid enough to withstand these horizontal pressures without material deformation, in their original state, this may not be the case after driving or jetting adjacent piles in the often confined space under a pier. It is believed that, in the case of solid upper layers, the transfer of horizontal forces may better be aided by other methods, such as projections below the pier base extending into excavated trenches in the foundation. In the case of soft upper layers, it is apparent that no great reliance can be placed on the resistance to horizontal forces except such as is furnished by the stiffness of the piles themselves.

It is the purpose of this paper to present, and encourage the use of, methods of design permitting the construction of pile foundations that transfer loads in any direction, including horizontal loads, to deep-lying foundation strata.

It may be argued, in favor of the common practice of relying on vertical piles to resist horizontal loads, that single test piles often show a remarkable resistance to horizontal pulls. However, it often happens, and may be expected, that the deflection of large pile foundations will materially exceed that of a single test pile with a comparable load. This is believed to be due to two main causes: One is the fact that the ground around the single test pile has not been disturbed by the driving or jetting of adjacent piles; the other is the fact, readily verified mathematically or experimentally, that a group consisting of many piles connected to a rigid pier will deflect far more if subjected to a horizontal loading than a single pile acted upon by a horizontal force equal to the average load on an individual pile in the group. In the investigation described in this paper, needless to say, it has been necessary to make certain simplifying assumptions. It may be argued, in general, that the assumptions are so many that the results of the investigation have little, if anything, to do with the actual conditions and, in particular, that considerations from the point of view of soil mechanics have been neglected in favor of those of purely structural mechanics. This argument cannot be refuted in a general manner. There may be cases in which the results are valueless because of the assumptions made. It is believed, however, that in the great majority of cases the simplifying assumptions will have only little effect on the final result. As has been the

case in other fields of engineering investigations, assumptions which are now thought necessary may gradually be eliminated by further investigation, so that ultimately a method of design will evolve that gives the entirely correct answer in all cases. Engineering progress is made in this manner.

NOTATION

The letter symbols adopted for this paper are defined where they first appear and are assembled for convenient reference in the Appendix.

GENERAL CONSIDERATIONS

Rational methods for the design of pile foundations that take into account the elastic deformations of the piles have been available for several years. That they are rarely used for practical design may be due to the fact that the general theory upon which they are based is somewhat difficult to follow and because the application to definite problems is often cumbersome. However,

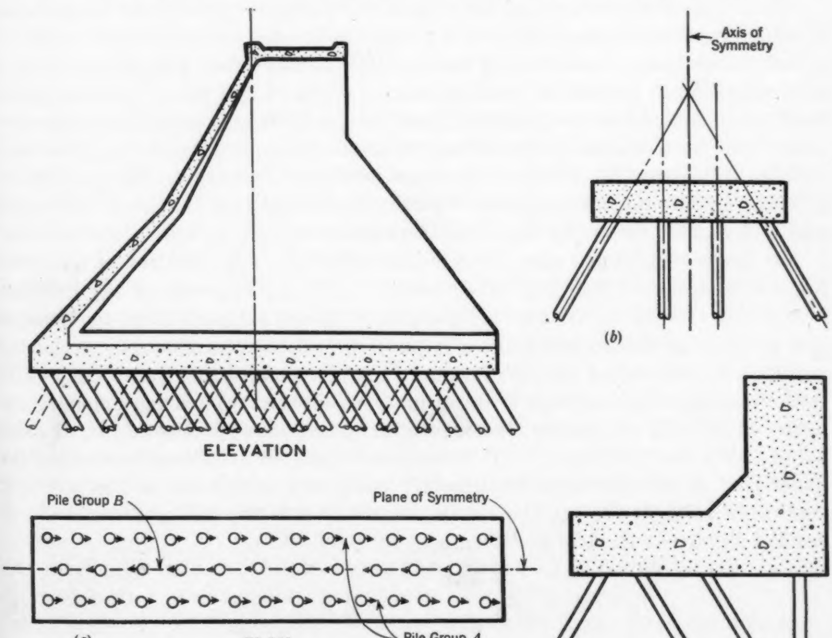

FIG. 1.—TYPES OF PILE FOUNDATIONS

in nearly all practical cases, the conditions are simplified to such an extent that neither the necessary theory nor application offers any particular difficulties. The investigation presented herein is confined to such simplified conditions. Those who are interested in the theory in its general form, or who have special problems to solve are referred to the original texts to which references are given

under the heading, "Acknowledgments." For the sake of clarity, the presentation, methods, and formulas differ somewhat from those of previous treatments of the subject. The basic principles, however, are not new and have been taken from the works cited.

The investigation is confined to two-dimensional systems; that is, to pile foundations having all piles parallel with, and symmetrical with respect to, a vertical plane of symmetry, and to structures in which the resultant of all outside forces, including dead loads, is located in the plane of symmetry. Although all piles are assumed parallel with this plane, they are not assumed to be parallel with each other. Three types of pile foundations are investigated: (*a*) Foundations consisting of two-pile groups, all piles in each group being parallel with each other (see Fig. 1(*a*)); (*b*) foundations having all piles symmetrical about a vertical line (pile bents) (see Fig. 1(*b*)); and (*c*) two-dimensional systems in general (piles in more than two directions and non-symmetrical) (see Fig. 1(*c*)).

It is assumed for all types that the structure on the piles (the pier) is infinitely stiff; that is, the internal deformation of the structure itself is disregarded.

Methods of procedure and formulas are first developed for the simple case in which each pile is considered as a column connected to the pier by means of a frictionless hinge, transmitting its load to the foundation material through a frictionless hinge located at some point of the axis of the pile. In many practical cases, such as bridge piers and gate piers in hydraulic structures, the error introduced by this assumption is insignificant. However, in some applications, such as pile bents for trestles, the error becomes of considerable magnitude. Therefore, methods are developed whereby the restraint of the pile heads in the pier and of the pile tips in the foundation material may be taken into account.

In the case of hinged piles the question arises as to the location of the lower hinge in the axis of the pile. Its location obviously depends on the material into which the pile is driven. If the pile is driven through loose material to firm ground the theoretical point of support would be approximately at the end point, and the so-called effective length of the pile would equal the actual length. If the pile is driven through uniform material so that its bearing capacity depends exclusively on friction, the theoretical point of support would be at some point above the pile tipe. If it is assumed that the frictional resistance per linear foot of pile increases rectilinearly from zero at the top of the pile to a maximum value at the tip, the elastic deformation of the pile due to a load, P, applied axially at the top is determined by:

$$\Delta = \frac{1}{E\,A} \int_0^L \left(P - \frac{1}{2} \times f_x \right) dx \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which x is a distance measured from the base of the pier to some arbitrary depth along the pile; f_x is the frictional resistance per linear foot at that point; E is the elastic modulus of the pile material; and A is the cross-section area of the pile. However, due to the assumed rectilinear variation,

$$f = \frac{x}{L} f_t \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which f_t is the frictional resistance per linear foot at the tip of the pile. Furthermore,

$$\frac{1}{2} f_t L = P \qquad (3)$$

since the total frictional resistance must equal the pile load. Hence,

$$\Delta = \frac{1}{E\,A} \int_0^L \left(P - \frac{x^2}{L^2} P \right) dx = \frac{1}{E\,A} \frac{2}{3} L\,P \qquad (4)$$

The deformation of a friction pile is thus the same as that of a pile of two-thirds the length, driven to firm ground, and its effective length is two-thirds of the actual length.

It is apparent that in the case of a pile, hinged both at its connection to the pier and at the point where it transfers its load to the foundation, the lateral resistance of the ground to movements perpendicular to the pile axis has been disregarded. In this case the exterior load on the pier is resisted solely by forces along the axes of the piles. On the other hand, there must be lateral resistance if the pile is considered restrained at one end or at both ends. The resistance must depend on the magnitude of the lateral displacement, on the depth below the surface, and, possibly, on several other factors, and it must be distributed in some manner along the entire length of the pile. For the sake of the investigation presented herein it is assumed that the total effect on the pier of the lateral resistance on the pile is represented by a lateral force (perpendicular to the pile axis) and a restraining moment, both acting at some point along the pile axis. Until the results of further research on the subject are available, the location of this point, the centroid of the lateral forces, can only be fixed by the judgment of the designer.

First, let it be assumed that all piles supporting the pier are hinged, and that the pier is acted upon by an exterior force in the plane of symmetry. The pier will then be displaced from its no-load position, due to the elasticity of the piles. The displacement may take the form of a translation or a rotation, or a combination of both. Since a translation may be defined as a rotation about some point at infinity, any small displacement may be considered a rotation. If the exterior force takes the form of a rotating moment, M (a force couple), the pier will rotate about a particular point, O, which is of special interest and is usually referred to as the elastic center of the pile group. If, on the other hand, the pier is acted upon by a resultant force going through Point O, the pier will rotate about a point at infinity; that is, the displacement will be a translation. This follows directly from Maxwell's general theorem of reciprocity.

Any arbitrary loading on the pier may be resolved into a force through the elastic center and a rotating moment; and, as has been shown, a force through this particular point will produce a translation and a moment will produce a rotation about the same point. Therefore, if the location of the elastic center, O, is known, and if the direction of the translation due to a given force through O has been determined, the problem of individual pile stresses is obviously reduced to finding the pile loads caused by a rotation about O and a translation in a given direction. In the investigation that follows the resultant of all stresses

in a pile at its junction with the pier is designated the pile load or the pile reaction. The method followed to determine these quantities differs somewhat according to the type of foundation and will be dealt with for each individual type.

Foundations Consisting of Two-Pile Groups

Hinged Piles.—Let the resultant of all outside forces, including dead load, be R and let the two-pile groups be A and B, in Fig. 1(*a*). All piles in each group are parallel with each other and are assumed to be of the same length. Piles of Group A need not necessarily be of the same length as the piles of Group B.

In order to determine the location of the elastic center, give the pier an arbitrary small translation in a direction perpendicular to the direction of the piles in Group B. How this translation is brought about is, for the present, immaterial. As a result of the translation no loads will be induced in the piles in Group B, and all the piles in Group A will have the same load. The resultant of all pile loads will be a force parallel to the piles in Group A and going through the center of gravity of this group. Furthermore, a force, equal in magnitude and location but opposite in direction to this force, acting on the pier will produce a translation perpendicular to the direction of the piles in Group B. Next, give the pier a translation perpendicular to the direction of the piles in Group A. The resultant of all pile loads induced by the translation will be a force parallel to the piles in Group B and going through the center of gravity of this group; and a force, acting on the pier, equal in magnitude and location, but opposite in direction to the resultant, will cause a translation perpendicular to the direction of the piles in Group A. Two lines of force have thus been determined. An outside force acting on the pier along either one of these lines will produce a translation perpendicular to the other. An outside force going through the point of intersection of the two lines may be resolved into components along the two lines. Since each of the components produces translation without rotation, the force itself must also produce translation without rotation and, by definition, the point of intersection must be the elastic center.

The following rule may be established:

Rule 1.—In a pile foundation consisting of piles in only two directions the elastic center is the point of intersection between lines drawn through the centers of gravity of the two-pile groups and parallel with the directions of the piles in the respective groups.

It is further evident that:

Rule 2.—An outside force through the elastic center will result in loads on the piles which may be determined by resolving the force into components parallel with the directions of the piles in the two groups and dividing the magnitude of the components by the number of piles in the respective groups.

Pile loads due to a rotating moment may be found as follows: The moment will cause a rotation designated $\Delta\phi$ about the elastic center (Fig. 2). As a result, the length of each pile will be changed by an amount,

$$\epsilon = \rho \, \Delta\phi \sin\gamma = r \, \Delta\phi \quad \text{.....................(5)}$$

in which ρ is the radius vector to the top of the pile and r, the perpendicular distance from the elastic center to the pile. The reaction exerted by the pile due to the change in length is:

$$P_M = \frac{E\,A}{L}\,\epsilon = \frac{E\,A}{L}\,r\,\Delta\phi \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

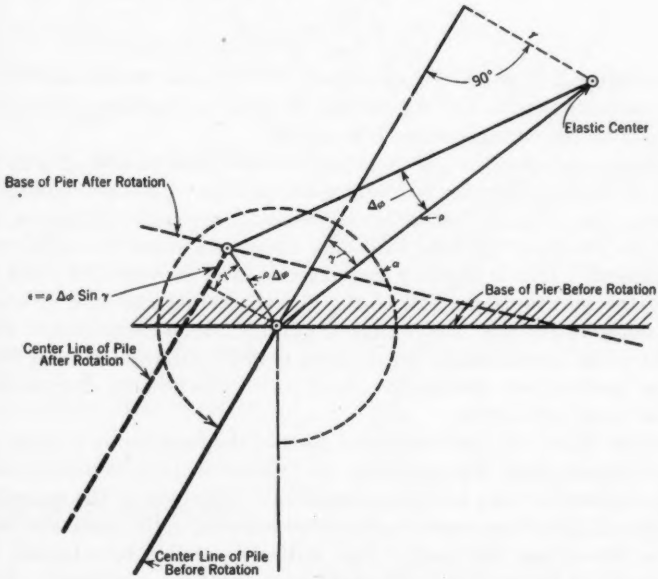

Fig. 2

The moment of all the pile reactions with respect to Point O must equal the rotating moment: $M = \sum \frac{E\,A}{L}\,r^2\,\Delta\phi$, or,

$$\Delta\phi = \frac{M}{\sum \frac{E\,A}{L}\,r^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The load on an individual pile is then determined by:

$$P_M = \frac{E\,A\,r}{L \sum \frac{E\,A}{L}\,r^2}\,M \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

If M is positive in the clockwise direction and compression is considered positive, r is positive if it extends from the point of rotation toward the right and negative if it extends from the point of rotation toward the left. In Fig. 2, r is negative.

If all piles have the same length, the same cross-sectional area, and are of the same material:

$$P_M = \frac{r}{\Sigma\,r^2}\,M \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Let the number of piles in Group A be n_A and in Group B, n_B, and let the component of the resultant in the direction of the A-piles be R_A and in the direction of the B-piles, R_B. The load on any pile may then be found from,

$$P_A = \frac{R_A}{n_A} + \frac{r}{\Sigma\, r^2} M \qquad (10a)$$

and,

$$P_B = \frac{R_B}{n_B} + \frac{r}{\Sigma\, r^2} M \qquad (10b)$$

The summation, $\Sigma\, r^2$, should be extended over both the A-piles and the B-piles and the components, R_A and R_B, should be taken as positive when they cause compression in the corresponding pile group.

For design purposes the pile foundations can often be arranged so that the moment, M, becomes zero for two principal loadings. In a hydraulic structure, for instance, the point of intersection between the resultant for maximum water load and the resultant for dead load only should be made the elastic center of the foundation. This is done by drawing two lines through the point of intersection of the resultants parallel to the piles in each group and arranging the piles in each group so that the centers of gravity fall on these lines. The loads for resultants for intermediate water levels may be determined from Equations (10). For preliminary designs the load determination for intermediate resultants is rarely necessary.

Restrained Piles.—If the number of piles in the foundation is great and the rotating moment small, the quantity, $\Delta\phi$ (Equation (7)), becomes small, and the effect of restraint may be safely neglected. However, if the quantity, $\Sigma\, r^2$, is small the rotation may cause considerable bending in the piles with resultant changes in the direct pile loads. It is entirely possible to calculate the pile stresses, both due to axial load and bending, under these conditions. It is done by adding "dummy" piles perpendicular to each of the real piles, the location and length of the dummy piles to be such that the effect on the pier of an unrestrained real pile plus one or two unrestrained dummy piles is the same as that of the restrained real pile. The dummy pile was first introduced by Dr. C. Nökkentved.[5] After the locations and lengths of the dummy piles have been determined, the procedure of load determination is exactly as if all piles were real and unrestrained. The dummy piles are thus purely imaginary piles added for the purpose of facilitating the computations. Certain loads will be found on the dummy piles, and these loads represent the actual shears in the real piles. The addition of the dummy piles is the method whereby the effect of the restraint of the real piles is introduced in the computations.

The restraint of the piles may be classified as follows:

1.—Piles Restrained in the Foundation Material at Some Point of Their Axis and Connected to the Pier Through a Hinge.—It can be shown that the total effect of the pile foundation on the pier is not changed by considering the real piles hinged at both ends, providing a hinged dummy pile is added perpendicular to each real pile, intersecting it at the base of the pier. The dummy pile should be given an area equal to that of the real pile and an effective length,

$$L_1 = \frac{1}{3}\frac{A}{I} L_r^3 \qquad (11)$$

where A and I represent the area and moment of inertia, respectively, of the real pile, and L_r, the distance from the base of the pier measured along the axis of the real pile to the point where it is considered restrained.

In Fig. 3 is indicated a pier and one pile, AE, of a pile group. Give the pier a small rotation, $\Delta\phi$, about the arbitrary point, O. As shown in Fig. 3, the shortening of the pile is the distance $A\,D$, or,

$$\epsilon = \rho\,\Delta\phi \sin\gamma = r\,\Delta\phi \quad (12)$$

Furthermore, the lateral deflection of the pile is $D\,B$, or,

$$\delta = \rho\,\Delta\phi \cos\gamma = a\,\Delta\phi \quad (13)$$

in which a = distance from the top of the pile, measured along the axis of the pile to a perpendicular to the axis through the elastic center. The deflection, δ, of the pile will cause a lateral reaction on the pier,

$$H = \frac{3\,E\,I}{L_r^3}\,\delta \quad (14)$$

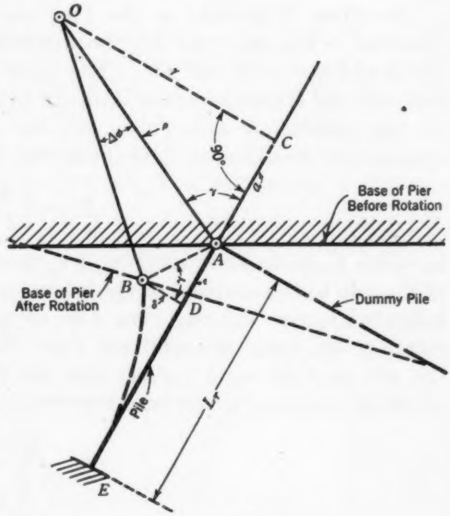

FIG. 3

in addition to the axial reaction:

$$V = \frac{E\,A}{L}\,\epsilon \quad (15)$$

Inserting the value of δ from Equation (13):

$$H = \frac{3\,E\,I}{L_r^3}\,a\,\Delta\phi \quad (16)$$

However, the lateral reaction, H, may also be obtained by adding a dummy pile perpendicular to AE and intersecting the pier base at Point A, provided the length of the dummy pile is,

$$L_1 = \frac{1}{3}\,\frac{A}{I}\,L_r^3 \quad (17)$$

The load on this pile due to the rotation, $\Delta\phi$, would be:

$$P_1 = \frac{E\,A}{L_1}\,\delta = \frac{E\,A}{L_1}\,a\,\Delta\phi \quad (18)$$

Inserting the value of L_1 from Equation (17):

$$P_1 = \frac{3\,E\,I}{L_r^3}\,a\,\Delta\phi = H \quad (19)$$

Since the point, O, is chosen in an entirely arbitrary manner, the substitution of the dummy pile for the restraining action of the real pile is obviously per-

missible for any displacement of the pier. The stress in the real pile, due to the restraint, is equal to that of a cantilever restrained at Point E and acted upon by a force, P_1, the axial load on the dummy pile, acting perpendicularly to the real pile at Point A.

2.—Piles Restrained at the Pier and not Restrained in the Foundation Material.—The real piles may be considered hinged at both ends if a dummy pile is added to each real pile. The dummy pile should be perpendicular to the real pile and should intersect it at the centroid of the lateral reactions exerted by the foundation material on the real pile. The cross-section area of the dummy pile should equal that of the real pile, and its effective length should be:

$$L_2 = \frac{1}{3}\frac{A}{I} L_r^3 \qquad (20)$$

in which L_r is the distance from the base of the pier, measured along the axis of the pile to the centroid of the lateral resistance in the ground. In Fig. 4 is indicated a pier and one pile, $A\,E$, of a pile group. Give the pier a small rotation, $\Delta\phi$, about the arbitrary Point O. Then, as before, the shortening of the pile is $A\,D$, or, $\epsilon = r\,\Delta\phi$; and the deflection is $D\,B$, or, $\delta = a\,\Delta\phi$. In addition thereto, the pile head receives a rotation equal to $\Delta\phi$.

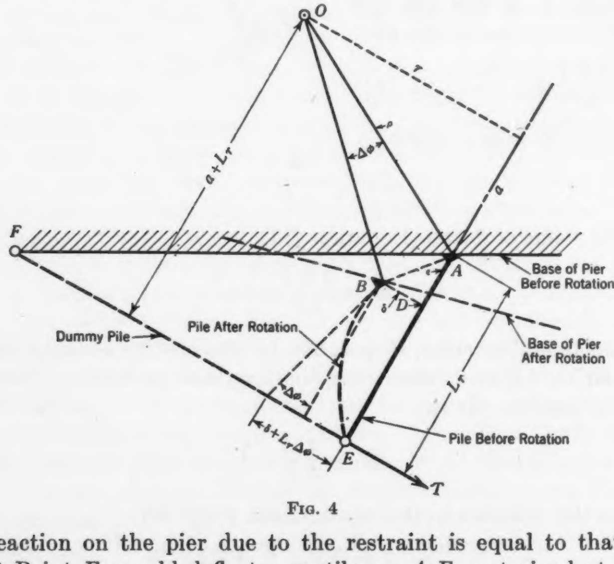

FIG. 4

The reaction on the pier due to the restraint is equal to that which, if applied at Point E, would deflect a cantilever, $A\,E$, restrained at Point A, a distance,

$$\delta_2 = \delta + L_r\,\Delta\phi \qquad (21)$$

The force required to produce a deflection, δ_2, is:

$$H = \frac{3\,E\,I}{L_r^3}\,\delta_2 = \frac{3\,E\,I}{L_r^3}\,(\delta + L_r\,\Delta\phi) = \frac{3\,E\,I}{L_r^3}\,\Delta\phi\,(a + L_r) \qquad (22)$$

The lateral reaction, H, may also be obtained by adding a hinged dummy pile

through Point E and perpendicular to $A\ E$, intersecting the pier base at Point F and having a length:

$$L_2 = \frac{1}{3}\frac{A}{I} L_r^3 \quad \text{.......................} (23)$$

The load on this pile, due to the rotation, $\Delta\phi$, would be:

$$P_2 = \frac{E\ A}{L_2}\Delta\phi\ (a + L_r) = \frac{3\ E\ I}{L_r^3}\Delta\phi\ (a + L_r) = H \quad \text{.........} (24)$$

The stress in the real pile, due to the restraint, is equal to that of a cantilever restrained at Point A and acted upon by a force, P_2, the axial load on the dummy pile, perpendicular to the real pile at Point E.

3.—Piles Restrained Both at the Pier and in the Foundation Material.—The real pile may be considered hinged at both ends if two dummy piles are added to each real pile. The dummy piles should be perpendicular to the real pile and should intersect it, respectively, at the base of the pier and at a distance below the base of the pier equal to two-thirds of the total distance from the base to the point of restraint in the foundation material. The areas of the dummy piles should equal the area of the real pile and the effective length of the dummy pile intersecting at the base:

$$L_1 = \frac{1}{3}\frac{A}{I} L_r^3 \quad \text{.......................} (25)$$

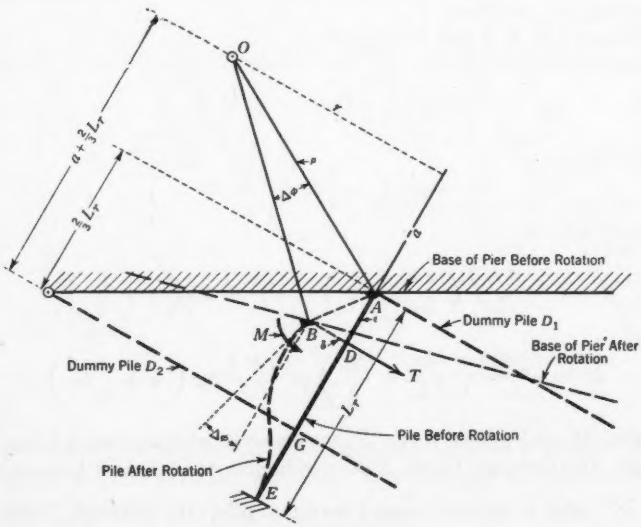

FIG. 5

and the effective length of the dummy pile intersecting at the two-thirds point:

$$L_2 = \frac{1}{9}\frac{A}{I} L_r^3 \quad \text{.......................} (26)$$

Since the two dummy piles are parallel, they will receive the same deformation for any translation and, for determining the elastic center only, may be combined into a single dummy pile with the same cross-section area as the real pile and an effective length:

$$L' = \frac{1}{12} \frac{A}{I} L_r^3 \quad \text{.....................} (27)$$

The combined dummy pile should intersect the real pile at a distance below the base of the pier equal to one-half the total distance from the base to the point of restraint. After the elastic center has been located, the two individual dummy piles should be used for the determination of pile loads. In Fig. 5 is indicated a pier and one pile, $A\ E$, of a pile group. The pile is restrained at both ends. Give the pier a small rotation, $\Delta\phi$, about the arbitrary point, O. Then the shortening of the pile, $A\ D$, is $\epsilon = r\ \Delta\phi$; and the deflection, $D\ B$, is $\delta = a\ \Delta\phi$. In addition, the pile head received a rotation equal to $\Delta\phi$.

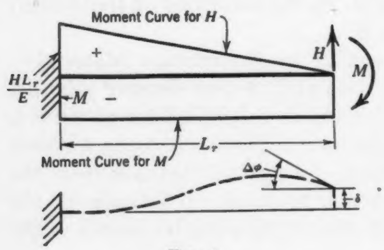

FIG. 6

The effect of the restraint on the pier would be a force, H, and moment, M, equal and opposite to a force and a moment which would produce, in a beam, $A\ E$, restrained at E, a deflection, δ, and a rotation, $\Delta\phi$, at Point A.

Considering Fig. 6 it will be seen that,

$$\delta = \frac{1}{E\ I}\left(\frac{1}{3} H\ L_r^3 - \frac{1}{2} M\ L_r^2\right) \quad \text{..............} (28a)$$

and,

$$\Delta\phi = \frac{1}{E\ I}\left(-\frac{1}{2} H\ L_r^2 + M\ L_r\right) \quad \text{..............} (28b)$$

Solving for H and M:

$$H = \frac{6\ E\ I}{L_r^2}\left(\frac{2\ \delta}{L_r} + \Delta\phi\right) = \frac{12\ E\ I}{L_r^3} \Delta\phi \left(a + \frac{L_r}{2}\right) \quad \text{.......} (29a)$$

and,

$$M = \frac{12\ E\ I}{L_r}\left(\frac{\delta}{2\ L_r} + \frac{\Delta\phi}{3}\right) = \frac{6\ E\ I}{L_r^2} \Delta\phi \left(a + \frac{2}{3} L_r\right) \quad \text{......} (29b)$$

The moment, M, and lateral force, H, may also be obtained by adding a hinged dummy pile, D_1, through Point A perpendicular to $A\ E$ and having a length: $L_1 = \frac{1}{3} \frac{A}{I} L_r^3$; and a second hinged dummy pile, D_2, through Point G, perpendicular to $A\ E$, intersecting the pier at Point F, and having the length: $L_2 = \frac{1}{9} \frac{A}{I} L_r^3$. The load on Pile D_1 due to the rotation would be,

$$P_1 = \frac{E\ A}{L_1} \Delta\phi\ a = \frac{3\ E\ I}{L_r^3} a\ \Delta\phi \quad \text{................} (30a)$$

and the load on Pile D_2:

$$P_2 = \frac{E\,A}{L_1}\Delta\phi\left(a + \frac{2}{3}L_r\right) = \frac{9\,E\,I}{L_r^3}\Delta\phi\left(a + \frac{2}{3}L_r\right) \quad \text{(30b)}$$

The sum of Equations (30a) and (30b) is:

$$P_1 + P_2 = \frac{12\,E\,I}{L_r^3}\Delta\phi\left(a + \frac{1}{2}L_r\right) = H \quad \text{(31)}$$

The moment of P_1 with respect to Point A is zero and the moment of P_2 becomes:

$$P_2\frac{2}{3}L_r = \frac{6\,E\,I}{L_r^2}\Delta\phi\left(a + \frac{2}{3}L_r\right) = M \quad \text{(32)}$$

The stresses in the real pile, due to the restraint, may readily be determined by referring to Fig. 6 and utilizing the relationships, $H = P_1 + P_2$; and, $M = P_2 \frac{2}{3} L_r$. Since the dummy piles constitute a pile group in a third direction, the pile loads cannot ordinarily be found by the method outlined for hinged piles, in only the two directions. Other methods are developed elsewhere in this paper which will permit solutions for this case.

Pile Foundations Which Are Symmetrical About a Vertical Line

Hinged Piles.—To determine the elastic center, give the pier a small horizontal translation equal to unity and toward the right (Fig. 7(a)). As a

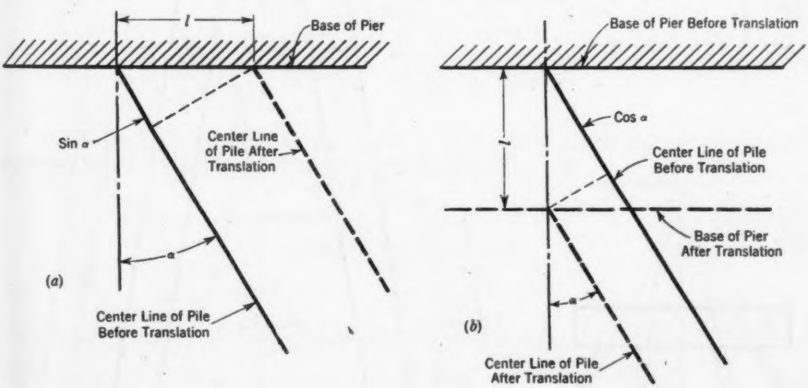

Fig. 7

result the length of each pile will be changed an amount $\epsilon = \sin\alpha$; in which α is the angle of the pile with the vertical. The reaction exerted by the pile due to the change in length is:

$$P_H' = \frac{E\,A\sin\alpha}{L} \quad \text{(33a)}$$

Due to symmetry, the resultant of the pile reactions must be a horizontal force:

$$R_H = \sum\frac{E\,A}{L}\sin^2\alpha \quad \text{(33b)}$$

Next give the pier a vertical translation, unity, downward (Fig. 7(*b*)). The change of length of each pile will be $\epsilon = \cos \alpha$; and the reaction:

$$P_V' = \frac{E\,A \cos \alpha}{L} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (34a)$$

Due to the symmetry, the resultant of the pile reactions must be a vertical force:

$$R_V = \sum \frac{E\,A}{L} \cos^2 \alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots (34b)$$

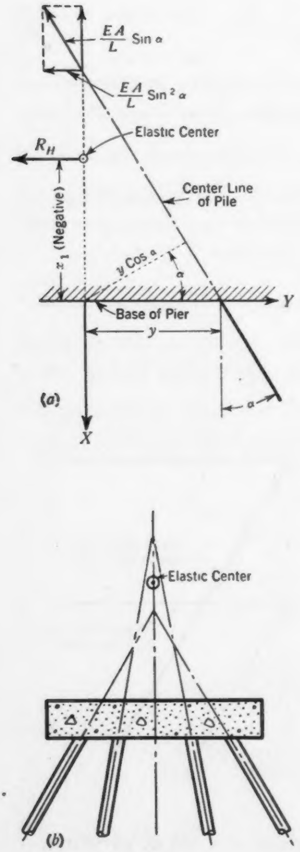

FIG. 8

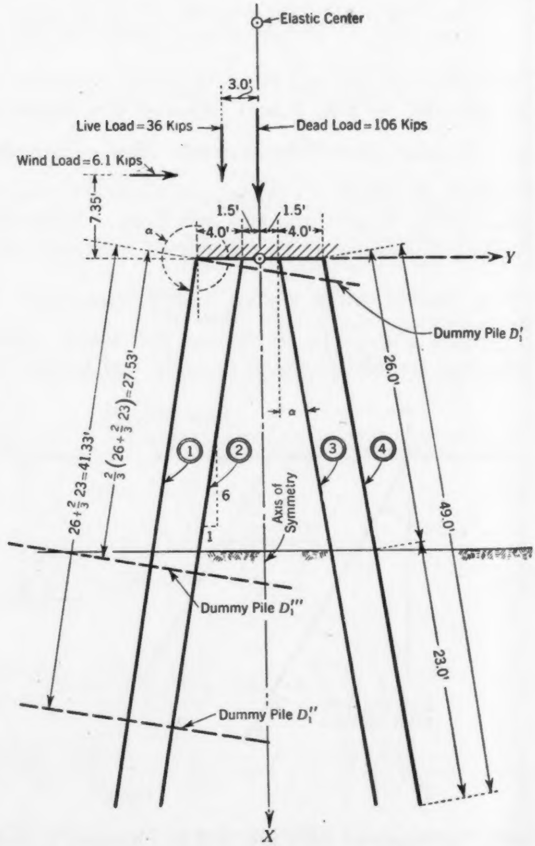

FIG. 9

located in the axis of symmetry. The location of the resultant, R_H, may be found graphically by drawing a force polygon and a string polygon to the forces expressed by Equation (33*a*).

It may also be found analytically as follows: Lay a co-ordinate system with a *X*-axis vertical along the axis of symmetry with positive direction downward, and a *Y*-axis horizontal, at the base of the pier, if this is horizontal,

with positive direction to the right. Let the distance from the point of origin of the co-ordinate system to the intersection of the piles with the Y-axis be y (Fig. 8(a)); then the X-co-ordinate to the elastic center is:

$$x_1 = -\frac{\sum \frac{E\,A}{L}\,y \sin\alpha \cos\alpha}{\sum \frac{E\,A}{L} \sin^2\alpha} \qquad (35a)$$

If all piles have the same effective length and the same cross-section area, and are of the same material:

$$x_1 = -\frac{\Sigma\, y \sin\alpha \cos\alpha}{\Sigma \sin^2\alpha} \qquad (35b)$$

The elastic center is thus a point on the symmetry axis having an X-co-ordinate of x_1. If a horizontal force, equal in magnitude and location to R_H but in the opposite direction, will cause a translation of unity and a pile load, P_H', then a horizontal force, H', through the elastic center will cause a pile load:

$$P_H = \frac{H'}{R_H} P_H' = \frac{\frac{E\,A}{L} \sin\alpha}{\sum \frac{E\,A}{L} \sin^2\alpha} H' \qquad (36a)$$

Similarly, a vertical force, V', through the elastic center will cause a pile load:

$$P_V = \frac{V'}{R_V} P_V' = \frac{\frac{E\,A}{L} \cos\alpha}{\sum \frac{E\,A}{L} \cos^2\alpha} V' \qquad (36b)$$

By the same reasoning as was used for pile foundations having piles in only two directions, the pile load due to a rotating moment is:

$$P_M = \frac{\frac{E\,A}{L}\,r}{\sum \frac{E\,A}{L}\,r^2} M \qquad (37)$$

The total pile load is then $P = P_H + P_V + P_M$; or,

$$P = \frac{\frac{E\,A}{L} \sin\alpha}{\sum \frac{E\,A}{L} \sin^2\alpha} H' + \frac{\frac{E\,A}{L} \cos\alpha}{\sum \frac{E\,A}{L} \cos^2\alpha} V' + \frac{\frac{E\,A}{L}\,r}{\sum \frac{E\,A}{L}\,r^2} M \qquad (38a)$$

If all piles have the same length and the same cross-section area and are of the same material,

$$P = \frac{\sin\alpha}{\Sigma \sin^2\alpha} H' + \frac{\cos\alpha}{\Sigma \cos^2\alpha} V' + \frac{r}{\Sigma\, r^2} M \qquad (38b)$$

The positive direction of M has been given previously. The positive direction of H' is toward the right; and the positive direction of V' is downward. The

positive direction of α is from the downward direction of the vertical, counterclockwise to the downward direction of the pile axis.

Comment.—Some interesting conclusions may be drawn from Equations (25) and (26). Since vertical piles do not help to resist a horizontal external force ($\alpha = 0$), structures subject to large horizontal forces should have as many battered piles as possible and the angle, α, should be made as great as possible. In structures where the rotating moment, M, is large, the quantity, $\Sigma\, r^2$, should be as large as possible. If $\Sigma\, r^2$ approaches zero the pile loads obviously approach infinity unless M is also zero. From Equation (7) it is seen that the rotation due to a moment increases as the quantity, $\Sigma\, r^2$, decreases; if, therefore, the piles, approximately, go through the same point the rotation for a given moment would be large. The assumption that the effect of restraint may safely be neglected does not hold for this case and the stability of the pier depends almost entirely on the restraint and the stiffness of the piles.

It is worth noting that in one of the most commonly used pile bents (Fig. 8(*b*)) the center lines of the piles come very close to intersection at one point. Large displacements may be expected in a structure of this construction if it is subject to loads that do not pass through the elastic center.

Restrained Piles.—The restraint of the piles may be taken into account by the methods outlined previously. It should be noted that the addition of the dummy piles does not destroy the original symmetry so that after they have been added, the computations are exactly as given for hinged piles.

Two-Dimensional Systems in General

The procedure is very similar to that used for symmetrical pile groups. Give the pier an arbitrary, small horizontal translation, unity, toward the right. The reactions exerted by the piles due to the translation is given by Equation (33*a*). The location of the resultant of all the pile reactions may be found graphically by drawing a force polygon and a string polygon to the forces, P_H'. Let the resultant of these reactions due to a unit translation be R_H. If the pier is acted upon by an outside force, H', in the same location as R_H, but acting in the opposite direction, the resultant pile loads would be,

$$P_H = \frac{H'}{R_H} P_H' = \frac{E\,A}{L} \sin\alpha \frac{H'}{R_H} \ldots\ldots\ldots\ldots\ldots (39)$$

Since R_H is determined by graphical construction, P_H can be found for any value of H'. Next, give the pier an arbitrary, small vertical translation, unity, downward. The reactions exerted by the piles due to the translation is then given by Equation (34*a*). The location of the resultant of all the pile reactions may be found graphically by drawing a force polygon and a string polygon to the forces, P_V'. Let the resultant of these reactions, due to a unit translation be R_V. As a check it should be noted that the vertical projection of R_H equals the horizontal projection of R_V.

If the pier is acted upon by an outside force, V', in the same location as R_V, but acting in the opposite direction, the resultant pile reactions will be,

$$P_V = \frac{V'}{R_V} P_V' = \frac{E\,A}{L} \cos\alpha \frac{V'}{R_V} \ldots\ldots\ldots\ldots\ldots (40)$$

Since R_V is determined by graphical construction, P_V can be found for any value of V'. It follows from the determination of the forces, R_H, and R_V, that the point of intersection between them is the elastic center for the pile foundation. A moment, therefore, will produce rotation about this point and loads on the individual piles determined by Equation (37). The total load on a pile due to an outside force, with components, H' and V', in the directions opposite to R_H and R_V, respectively, and with a moment, M, about the elastic center is:

$$P = P_H + P_V + P_M = \frac{E\,A}{L}\left[\frac{H'}{R_H}\sin\alpha + \frac{V'}{R_V}\cos\alpha + \frac{r}{\sum \frac{E\,A}{L} r^2} M\right] \quad \text{..(41)}$$

The positive directions for M, α, and r have been defined previously. The positive directions for H' and V' are opposite to the directions of R_H and R_V.

If all piles are of the same length and the same material, the string and force polygons should be drawn to the forces: $P_H'' = \sin\alpha$; and, $P_V'' = \cos\alpha$, corresponding to the horizontal or vertical translations, $\Delta = \frac{L}{E\,A}$. Let the resultants of the pile reactions due to these translations be R_H' and R_V', respectively. If the pier is acted upon by outside forces, H' and V', in the same location but acting in the opposite direction of R_H' and R_V', the pile loads would be: $P_H = \frac{H'}{R_H'}\sin\alpha$; and, $P_V = \frac{V'}{R_V'}\cos\alpha$. The total pile load, in case all piles are of the same length and same material, is then,

$$P = \frac{H'}{R_H'}\sin\alpha + \frac{V'}{R_V'}\cos\alpha + \frac{r}{\Sigma r^2} M \quad \text{..............(42)}$$

It should be noted that in this general case the forces, R_H, R_H', R_V, and R_V', are not horizontal and vertical forces. They are the resultants of pile reactions due to horizontal and vertical translations. Only in the case of symmetrical pile foundations do they become horizontal and vertical forces.

Numerical Example

On Fig. 9 is shown a symmetrical pile bent consisting of four piles driven at a batter of 1 : 6 and spaced as shown. The piles are of reinforced concrete, 49 ft long, and driven 23 ft into fine sand. The cross-section areas of all piles are the same, and equal 331 sq in.; the moments of inertia of the cross-sections are 8 800 in.[4] The effective lengths are $49 - 23 + \frac{2}{3}23 = 41.33$ ft. The pile bent is subjected to three loads: A dead load of 106 kips acting vertically in the axis of symmetry; a live load of 36 kips acting vertically 3.0 ft from the axis; and a wind load of 6.1 kips acting horizontally 7.35 ft above the base of the pier.

The pile loads are determined under the assumptions that all piles are: (1) Hinged both at the pier and in the foundation; (2) hinged at the pier and restrained in the foundation material; (3) restrained at the pier and hinged in the foundation material; or, (4) restrained both at the pier and in the foundation material.

The piles are numbered 1 to 4, inclusive, from left to right; the dummy piles for Assumption (2) are designated D_1' to D_4'; for Assumption (3), D_1'' to D_4''; and for Assumption (4), D_1' to D_4' at the base of the pier and D_1''' to D_4''' at the two-thirds point.

TABLE 1.—NUMERICAL VALUES USED IN ILLUSTRATIVE EXAMPLE

Pile No.	Distance, y, in feet (1)	Angle, α (2)	$\sin \alpha$ (3)	$\cos \alpha$ (4)	$\sin^2 \alpha$ (5)	$\cos^2 \alpha$ (6)	$y \sin \alpha \cos \alpha$ (7)
1	−5.50	−9°28′	−0.164	0.986	0.0269	0.972	+0.890
2	−1.50	−9°28′	−0.164	0.986	0.0269	0.972	+0.243
3	+1.50	+9°28′	+0.164	0.986	0.0269	0.972	+0.243
4	+5.50	+9°28′	+0.164	0.986	0.0269	0.972	+0.890
D_1'	−5.50	+80°32′	+0.986	0.164	0.972	0.0269	−0.890
D_2'	−1.50	+80°32′	+0.986	0.164	0.972	0.0269	−0.243
D_3'	+1.50	−80°32′	−0.986	0.164	0.972	0.0269	−0.243
D_4'	+5.50	−80°32′	−0.986	0.164	0.972	0.0269	−0.890
D_1''	−256.9	+80°32′	+0.986	0.164	0.972	0.0269	−41.54
D_2''	−252.9	+80°32′	+0.986	0.164	0.972	0.0269	−40.89
D_3''	+252.9	−80°32′	−0.986	0.164	0.972	0.0269	−40.89
D_4''	+256.9	−80°32′	−0.986	0.164	0.972	0.0269	−41.54
D_1'''	−173.1	+80°32′	+0.986	0.164	0.972	0.0269	−27.99
D_2'''	−169.1	+80°32′	+0.986	0.164	0.972	0.0269	−27.34
D_3'''	+169.1	−80°32′	−0.986	0.164	0.972	0.0269	−27.34
D_4'''	+173.1	−80°32′	−0.986	0.164	0.972	0.0269	−27.99

TABLE 1.—(*Continued*)

Pile No.	L (8)	$\frac{10^4 \sin \alpha}{L}$ (9)	$\frac{10^4 \cos \alpha}{L}$ (10)	$\frac{10^4 \sin^2 \alpha}{L}$ (11)	$\frac{10^4 \cos^2 \alpha}{L}$ (12)	$\frac{10^4 y \sin \alpha \cos \alpha}{L}$ (13)
1	41.33	−39.68	238.57	6.51	235.18	+215.34
2	41.33	−39.68	238.57	6.51	235.18	+58.80
3	41.33	+39.68	238.57	6.51	235.18	+58.80
4	41.33	+39.68	238.57	6.51	235.18	+215.34
D_1'	128 100	+0.08	0.01	0.08	0.00	−0.07
D_2'	128 100	+0.08	0.01	0.08	0.00	−0.02
D_3'	128 100	−0.08	0.01	0.08	0.00	−0.02
D_4'	128 100	−0.08	0.01	0.08	0.00	−0.07
D_1''	128 100	+0.08	0.01	0.08	0.00	−3.24
D_2''	128 100	+0.08	0.01	0.08	0.00	−3.19
D_3''	128 100	−0.08	0.01	0.08	0.00	−3.19
D_4''	128 100	−0.08	0.01	0.08	0.00	−3.24
D_1'''	42 700	+0.23	0.04	0.23	0.01	−6.55
D_2'''	42 700	+0.23	0.04	0.23	0.01	−6.40
D_3'''	42 700	−0.23	0.04	0.23	0.01	−6.40
D_4'''	42 700	−0.23	0.04	0.23	0.01	−6.55

Table 1 contains all numerical values used in the computations, including the lengths of the dummy piles calculated from Equations (11), (20), (25), and (26). Table 2(*a*) gives the computations of pile loads under Assumption (1). The location of the elastic center is found from Equation (35*b*) and the pile loads from Equation (38*b*). Table 2(*b*) gives the computations of pile loads under Assumption (2). The location of the elastic center is found from Equation (35*a*), and the pile loads from Equation (38*a*). Since all piles have the same cross-section and are of the same material, the quantity, *EA*, cancels out in these two equations. Tables 2(*c*) and 2(*d*) give the corresponding computations under Assumptions (3) and (4), respectively. Zero load on a dummy pile indicates that the load is negligible.

TABLE 2.—ILLUSTRATIVE EXAMPLE (LOADS ARE IN KIPS)

Pile No.	Distance, r, in feet	Values of r^2	$10^4 \frac{r}{L}$	$10^4 \frac{r^2}{L}$	LIVE LOADS			Dead load,	WIND LOADS			Total,
					P_V	P_M	P	$P_V = P$	P_H	P_M	P	P
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
(*a*) COMPUTATIONS UNDER ASSUMPTION (1), $x_1 = -21.00$												
1......	−1.97	3.89	..	..	9.1	+13.7	+22.8	+26.9	−9.3	+10.5	+1.2	+50.9
2......	+1.97	3.89	..	..	9.1	−13.7	−4.6	+26.9	−9.3	−10.5	−19.8	+2.5
3......	−1.97	3.89	..	..	9.1	+13.7	+22.8	+26.9	+9.3	+10.5	+19.8	+69.5
4......	+1.97	3.89	..	..	9.1	−13.7	−4.6	+26.9	+9.3	−10.5	−1.2	+21.1
(*b*) COMPUTATIONS UNDER ASSUMPTION (2), $x_1 = -20.81$												
1......	−2.00	4.00	−483.9	967.8	+9.1	+13.4	+22.5	+26.9	−9.2	+10.2	+1.0	+50.4
2......	+1.94	3.77	+469.4	912.2	+9.1	−13.0	−3.9	+26.9	−9.2	−9.9	−19.1	+3.9
3......	−1.94	3.77	−469.4	912.2	+9.1	+13.0	+22.1	+26.9	+9.2	+9.9	+19.1	+68.1
4......	+2.00	4.00	+483.9	967.8	+9.1	−13.4	−4.3	+26.9	+9.2	−10.2	−1.0	+21.6
D_1'....	−21.42	458.82	−1.7	35.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
D_2'....	−20.77	431.39	−1.6	33.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
D_3'....	+20.77	431.39	+1.6	33.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
D_4'....	+21.42	458.82	+1.7	35.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(*c*) COMPUTATIONS UNDER ASSUMPTION (3), $x_1 = -20.31$												
1......	−2.08	4.33	−503.3	1 047.7	+9.1	+10.9	+20.0	+26.9	−9.2	+8.0	−1.2	+45.7
2......	+1.86	3.47	+450.0	839.6	+9.1	−9.8	−0.7	+26.9	−9.2	−7.2	−16.4	+9.8
3......	−1.86	3.47	−450.0	839.6	+9.1	+9.8	+18.9	+26.9	+9.2	+7.2	+16.4	+62.2
4......	+2.08	4.33	+503.3	1 047.7	+9.1	−10.9	−1.8	+26.9	+9.2	−8.0	+1.2	+26.3
D_1''....	−62.26	3 876	−4.9	302.6	0.0	+0.1	+0.1	0.0	0.0	+0.1	+0.1	+0.2
D_2''....	−61.61	3 796	−4.8	296.3	0.0	+0.1	+0.1	0.0	0.0	+0.1	+0.1	+0.2
D_3''....	+61.61	3 796	+4.8	296.3	0.0	−0.1	−0.1	0.0	0.0	−0.1	−0.1	−0.2
D_4''....	+62.26	3 876	+4.9	302.6	0.0	−0.1	−0.1	0.0	0.0	−0.1	−0.1	−0.2
(*d*) COMPUTATIONS UNDER ASSUMPTION (4), $x_1 = -19.14$												
1......	−2.28	5.20	−551.7	1 258.2	9.1	+9.9	+19.0	+26.9	−8.9	+6.6	−2.3	+43.6
2......	+1.66	2.76	+401.6	667.8	9.1	−7.2	+1.9	+26.9	−8.9	−4.8	−13.7	+15.1
3......	−1.66	2.76	−401.6	667.8	9.1	+7.2	+16.3	+26.9	+8.9	+4.8	+13.7	+56.9
4......	+2.28	5.20	+551.7	1 258.2	9.1	−9.9	−0.8	+26.9	+8.9	−6.6	+2.3	+28.4
D_1'....	−19.79	391.6	−1.5	30.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
D_2'....	−19.13	366.0	−1.5	28.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
D_3'....	+19.13	366.0	+1.5	28.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
D_4'....	+19.79	391.6	+1.5	30.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
D_1'''...	−47.32	2 239	−11.1	524.4	0.0	+0.2	+0.2	0.0	+0.1	+0.1	+0.2	+0.4
D_2'''...	−46.67	2 178	−10.9	510.1	0.0	+0.2	+0.2	0.0	+0.1	+0.1	+0.2	+0.4
D_3'''...	+46.67	2 178	+10.9	510.1	0.0	−0.2	−0.2	0.0	−0.1	−0.1	−0.2	−0.4
D_4'''...	+47.32	2 239	+11.1	524.9	0.0	−0.2	−0.2	0.0	−0.1	−0.1	−0.2	−0.4

A study of the pile loads due to the different external forces will reveal several interesting facts. Under all assumptions, Pile No. 2 gets tension for live load only; it is doubtful whether this result would have been anticipated had other, less rational, methods of load determination been used. Restraint in the foundation material only, has little effect on the pile loads. The effect of restraint in the pier is more pronounced and the effect of fully restrained piles is considerable. Had the horizontal distances between the piles been greater, the effect of all restraints would have been decreased.

ACKNOWLEDGMENTS

The writer wishes to acknowledge the valuable assistance of Mr. W. A. Larsen and the constructive criticism contributed by him and other engineers

of the United States Bureau of Reclamation in connection with the preparation of this paper. The investigation was conducted in connection with the design of the Imperial Dam, of the Boulder Canyon Project.

He has also consulted, freely, the fundamental work done in this field of study by Messrs. T. Hultin,[2] P. Gullander,[3] H. M. Westergaard,[4] M. Am. Soc. C. E., and C. Nökkentved.[5] In addition the following investigators have treated various phases of the subject: A. Ostenteld,[6] H. Wünch,[7] T. Schultze,[8] A. Labutin,[9] P. Hedde,[10] and Franx.[11]

APPENDIX

NOTATION

The following symbols, defined where first introduced in the paper, are rearranged herein for convenience of reference. An effort has been made to conform essentially with "Symbols for Mechanics, Structural Engineering, and Testing Material,"[12] compiled by a committee of the American Standards Association, with Society representation, and approved by the Association in 1932:

- A = cross-section area of a pile;
- a = distance from the top of a pile, measured along the axis of the pile, to a perpendicular to the axis through the elastic center.
- E = modulus of elasticity of the pile material;
- f = a function factor; the frictional resistance per linear foot of pile; f_x = the resistance at distance, x; and, f_t = the resistance at the tip of the pile;
- H = a lateral reaction; H' = a horizontal force;
- I = moment of inertia;
- L = effective length of a pile; L_r = distance from base of pier measured along axis of pile to point where it is considered restrained;
- M = a moment; a force couple;
- n = a number; n_A = a number of piles in Group A;
- P = a concentrated load; P_M = an additional load imposed by a change in pile length;
- R = resultant of all external forces; R_H = a horizontal force, the resultant of pile reactions; R_V = a vertical force, the resultant of pile reactions;

[2] "Om beräkning ov Grundpålninger," by T. Hultin, Industritidningen, *Norden*, 1911.
[3] "Teori för Grundpålninger," by P. Gullander, Stockholm, 1914.
[4] "The Resistance of a Group of Piles," by H. M. Westergaard, *Journal*, Western Soc. of Engrs., 1917.
[5] "Beregning af Pälevärker," by C. Nökkentved, Copenhagen, 1924.
[6] *Teknisk Tidskrift*, Copenhagen, 1921, No. 1.
[7] "Statische Berechnung der Pfalsysteme" Stuttgart, 1927.
[8] *Zentralblatt der Bauverwaltung*, 1926, p. 469.
[9] "Die graphische Berechnung von Pfalvosten, etc." Riga, 1933.
[10] *Der Bauingenieur*, 1929, p. 1.
[11] *De Ingenieur*, 1928, p. B-189.
[12] A.S.A.—Z10*a*—1932.

r = distance from the elastic center to the axis of the pile;
V = axial reaction; V' = a vertical reaction;
x = a depth from the base of the pier to an arbitrary point along the pile axis;
y = the distance from the point of origin of the co-ordinate system to the intersection of the piles with the Y-axis;
α = an angle (see Fig. 2) of the pile with the vertical;
γ = an angle (see Fig. 2);
Δ = the elastic deformation of a pile due to a load, P, applied axially at the top;
ϵ = the change in length of pile;
ρ = a radius vector from the elastic center to the top of a pile;
ϕ = a rotation; $\Delta\phi$ = a small rotation;
δ = deflection of a pile.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

FLOOD-PROTECTION DATA[1]

PROGRESS REPORT OF THE COMMITTEE

The disastrous floods of 1935, 1936, and 1937 have proved a blessing in disguise in stimulating much needed interest in flood data. This is clearly shown by the excellence and volume of the many investigations made and of the reports thereon. In consequence, flood records to-day are more detailed and trustworthy in regard to providing working data for engineers, than ever before. An outstanding example of the desire to authenticate and preserve for future generations the factual data made available by a great flood, is the remarkable work done by the State of Massachusetts during and immediately following the flood of March, 1936. The results of this work, which included the identifying and marking on permanent structures of 1 390 high-water marks along 27 rivers, are published in a report[2] of the Massachusetts Geodetic Survey, dated November, 1936. This undertaking was sponsored by the Massachusetts Department of Public Works and was financed with funds provided by the Works Progress Administration (WPA). The promptness with which the field work was started enabled its completion before the evidences left by the flood had been obliterated by succeeding rains. The sea-level elevation of each high-water mark was determined by reference to geodetic survey bench-marks. Aerial photographs were used extensively and the boundaries of overflowed areas were traced out and marked on maps. The report includes 56 river profiles and 49 sectional maps showing flood zones.

Work along similar lines was done in other New England States by the Corps of Engineers, U. S. Army, and by local organizations. The results, as published by the United States Geological Survey,[3] constituted a comprehensive compilation of data brought together from various sources, prepared in coöperation with the Federal Emergency Administration of Public Works as Part 1 of a series of three reports. Parts 2 and 3, still in preparation, cover, respectively, the Hudson River to the Susquehanna River Region[4] and the Potomac, James, and Upper Ohio Rivers.[5] These reports are noteworthy departures from previous flood reports in the matter of the abundance and the detail of the observed data presented. In Part 1,[3] 60 pages of tabular matter

[1] Presented at the Annual Meeting, New York, N. Y., January 19, 1938.

[2] "High-Water Data—Flood of March, 1936, in Massachusetts," by E. C. Houdlette, 1936.

[3] "The Floods of March, 1936—New England Rivers," *Water Supply Paper No. 798*, U. S. Geological Survey.

[4] *Loc. cit., Water Supply Paper No. 700.*

[5] "The Floods of March, 1936—New England Rivers," *Water Supply Paper No. 800*, U. S. Geological Survey.

are devoted to listing observed flood heights along New England rivers, and 22 pages relate to historic floods and great storms. Noteworthy is the inclusion of stage and discharge data at frequent intervals suitable for platting flood graphs.

More than 100 technical reports and papers concerned with floods and flood control, all containing flood data, have appeared during 1937. The number is too great for enumeration in this report. Only a few which have attracted the special attention of the Committee because of the nature of the data contained in them will be referred to herein:

"Floods in Texas," by Robert L. Lowry, Jr., Assoc. M. Am. Soc. C. E.[6] (Contains tabulations of excessive rainfalls exceeding 10 in. in 24 hr; a tabulation of 152 major flood records; a tabulation of 165 major floods in the United States, exclusive of Texas; and graphs of flood run-offs experienced in Texas);

"Flood in the La Cañada Valley, California, 1934."[7] (In addition to data relating to rainfall and run-off a feature of this report is the information relating to heavy débris movement by cloudburst run-off);

"Flood on Republican and Kansas Rivers, May and June, 1935,"[8] by Robert Follansbee, and J. B. Spiegel, Members, Am. Soc. C. E. (Correlation of data drawn from many sources, including the United States Weather Bureau, Corps of Engineers, U. S. Army, and State engineers, was made by C. H. Pierce, M. Am. Soc. C. E. The report cites a number of historic floods, and contains interesting information relating to the meteorology of the storm which produced the flood, and to rates of flood-crest travel);

"Flow of the Rio Grande and Tributary Contributions,"[9] from San Marcial, N. Mex., to the Gulf of Mexico, for the year 1936. (Contains a compilation of floods at San Marcial and at El Paso, Tex., extended back to 1828. Of special interest are the diagrams showing rates of flood-crest travel for floods of various magnitudes. The hydrographic work of this Commission is of a high order and special attention is devoted to floods);

"The Floods of March 1936 in Pennsylvania," by the Division of Hydrography of the Department of Forests and Waters, Commonwealth of Pennsylvania, 1936. (Contains a valuable discussion of flood forecasting and also of flood frequency determinations. This report and the following one were prepared in co-operation with the U. S. Geological Survey);

"Stream Flow Records, 1936—Elevations of Major Floods," by the Division of Hydrography of the Department of Forests and Waters, Commonwealth of Pennsylvania, 1937;

"Stages and Discharge Observations—Lower Valley of the Mississippi River, January 1 to June 30, 1937," published by the Mississippi River Comission, Vicksburg, Miss. (The data presented constitute the most complete set of observations ever made on a great flood in this river. The maximum discharge at the crest of the 1937 flood approximated 2 000 000 cu ft per sec. The flood-flow observations cover a period of three months. Daily discharge meas-

[6] *Transactions*, Am. Geophysical Union, 1937, Pt. II, pp. 541–561.
[7] *Water Supply Paper 796-C*, U. S. Geological Survey, 1937.
[8] *Loc. cit., 796-B.*
[9] *Water Bulletin No. 6*, International Boundary Comm., United States and Mexico, 1937

urements were made at a score of stations on the main river. The work thus accomplished is noteworthy because of the amount of personnel, equipment, and river craft required for measuring the flow of the Lower Mississippi at many points. In addition, hundreds of gages were read daily, and at the crest of the flood many were read hourly, thus furnishing accurate data for constructing flood profiles for both rising and falling stages, and for determining the rate of flood-crest travel);

"A Meteorological Analysis of the Possibility of the Coincidence of Maximum Flood Discharges from the Ohio, Mississippi, and Missouri Rivers," by the Corps of Engineers, U. S. Army, and the U. S. Weather Bureau Co-Operative Studies. (Contains a discussion of air-mass analysis with special reference to "cold fronts" such as occurred at the time of the 1937 flood in the Ohio Valley) (not published);

"Some Flood Producing Storms of the Atlantic Seaboard," by Montrose W. Hayes and Horace R. Byers; also "Meteorological Conditions during the March 1936 and Other Notable Floods," by Horace R. Byers;[10]

"The Meteorology of Great Floods in the Eastern United States," by Charles F. Brooks and Alfred H. Thiessen;[11]

"Modernizing Headwater Forecasting," by Merrill Bernard, M. Am. Soc. C. E.,[12] Chief, River and Flood Division, U. S. Weather Bureau. (An outline of the newer methods that must supersede the old, if important localities are to receive forecasts of flood stages sufficiently far in advance to be of value in saving lives and property. The importance of increasing the number of rainfall stations and of providing recording and also transmitting equipment is stressed);

"The Colorado Delta," by Godfrey Sykes, Research Associate, Carnegie Institution of Washington; published jointly by the Institution and the American Geographical Society of New York, 1937. (Discusses among other things the marked effect which flood reduction by large dams has had on the channel of the Colorado River, more especially in its Delta);

"Magnitude and Frequency of Floods on Illinois Streams," by G. W. Pickels, M. Am. Soc. C. E.[13] (Devotes much space to frequency studies, and the methods of analysis in use); and,

"Characteristics of Floods in the Southern Rocky Mountain Region," by Royce J. Tipton, Assoc. M. Am. Soc. C. E.[14]

During the year a number of important reports bearing on floods were in preparation. The U. S. Geological Survey has in preparation, in addition to *Water Supply Papers Nos. 799*[4] and *800*[5]: "Major Texas Floods of 1935"; and, "Major Texas Floods of 1936." The U. S. Weather Bureau has in preparation "Ohio and Mississippi Flood of 1937," to appear as Supplement No. 37 of *Monthly Weather Review*. It is contributing also to *Water Supply Paper No. 800*[5] a section entitled: "Weather Associated with the Floods of March 1936."

During 1937, the Corps of Engineers, U. S. Army, was engaged on preliminary examinations and surveys authorized under the 1936 Flood Control Act.

[10] *Journal*, New England Waterworks Assoc., Vol. LI, No. 2, 1937.

[11] *The Geographical Review*, Vol. XXVII, No. 2, April, 1937.

[12] *Engineering News-Record*, December 16, 1937.

[13] *Bulletin No. 296*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[14] *Transactions*, Am. Geophysical Union, 1937, Pt. II, pp. 592–600.

Owing to restriction of funds only about 80 of the 223 surveys authorized were undertaken. Of these, 20 have been reported on unfavorably. A few survey reports have been submitted, but are not yet available in printed form. The remaining surveys are in progress at the present time.

Mention should be made also of the active interest in flood matters which the Boston Society of Civil Engineers continues to take, and its appointment of a committee which has a report in preparation. The Section of Hydrology, American Geophysical Union, likewise has appointed a special committee, composed almost entirely of members of the Society, to study Flood Wave Phenomena in Natural Channels. The U. S. Soil Conservation Service is engaged on a study of transformation and time of transit of flood waves from specially equipped water-shed in Ohio.

Publication of Flood Data.—In the opinion of this Committee the provision of suitable means for publishing flood data in systematic form remains an unsolved problem. Although the quantity as well as the excellence of the reports that have been published during 1937 is highly gratifying, the fact remains that there is to-day no officially designated fountain-head for factual data relating to floods. A glance over the list of reports issued reveals that an engineer in search of information on this subject must keep in contact with at least four Government bureaus and, in addition, with State agencies, universities, engineering magazines, engineering societies, and special organizations like the Mississippi River Commission, the International Boundary Commission (United States and Mexico), and the Massachusetts Geodetic Survey. The reports issued by these various agencies are limited as to supply and are soon out of print as well as out of date. To illustrate, attention is invited to the edition of *Water Supply Paper No. 771*, "Floods in the United States—Magnitude and Frequency," published in December, 1935, which was practically exhausted within a year after publication. The records contained in this report had been carried to include 1934, but in numerous cases stopped with 1933, and are now from 3 to 4 yr behind the time. Since its publication there have been witnessed some of the greatest floods in the annals of the United States. The preparation of this paper was sponsored by the National Resources Committee of the Federal Government, was prepared by engineers of the U. S. Geological Survey, with advisory co-operation extended by the Society and the American Geophysical Union, and its cost was met from funds supplied by the PWA. The time is ripe for the issuance of a sequel to *Water Supply Paper No. 771*, but if a combination of circumstances like the foregoing is required to produce it, the prospects are not encouraging.

Engineers and authorities concerned with weighty flood-control problems have good reasons for complaint at the absurdity of the present-day system which compels them to cast about in a dozen directions for information which, because of its inestimable value in the cause of protecting life and property, should, as a matter of common sense, be within their easy reach at all times.

A central agency is needed, preferably in some Federal bureau, to act as a permanent source of information for flood data—published as well as unpublished—and to act as a clearing house for factual data pertaining to floods. Until an agency of this kind is created through Congressional or Administrative

action, and is given a permanent status and suitable appropriations, so long will engineers and the public at large remain in the present quandary as to whom to turn for data.

On May 24 to 26, 1937, the Committee met in Washington, D. C., and held conferences with the principal Government officials whose bureaus are engaged in the study of floods and means for their control, and who issue reports for general distribution in printed form. These included: The Chief Hydraulic Engineer, U. S. Geological Survey; the Chief, U. S. Weather Bureau; the Co-Ordinator of Flood-Control Activities of the U. S. Department of Agriculture (Bureau of Agricultural Economics); the Assistant Chief of Engineers, U. S. Army; and the Secretary of the Smithsonian Institution. The possibilities of their undertaking the systematic publication of flood data were discussed in each instance. In all cases, the desirability of such a plan was acknowledged, but lack of funds or lack of suitable authority was found to stand in the way of its accomplishment.

Meteorology of Floods.—This is a subject of considerable importance in which as yet little headway of a practical nature has been made. The meteorological conditions which gave rise to the January–February, 1937, flood in the Ohio Valley focused attention on the part of engineers to the necessity of gaining a clear understanding regarding air-mass analysis and more especially the occurrence of so-called "Arctic-air fronts" which cause the condensation of moist warm air ascending over them, thereby producing rains of great intensity as well as of great duration. To what extent "cold fronts" should be accepted as the primary cause of maximum flood conditions in any and all sections of the United States remains to be determined. The record flood of 1937 in the Ohio Valley resulted from 16 to 22 in. of rain falling in 26 days. By contrast, in Texas, the greatest recorded floods have been caused by 20 in. of rain in from 24 to 48 hr over areas far too large to fall in the category of cloudbursts and without cold frontal action. In the Miami Valley, Ohio, the flood of March, 1913, caused by a cold front, produced from 8 to 10 in. of rain over a period of 5 days, and the flood-control works built there were designed to take care of 14 in. in 5 days, then believed to be on the side of safety for that latitude. In New York State, record floods in July, 1935, were caused by 7 to 14 in. in 16 hr. Along the Gulf Coast, 12 in. in 24 hr has repeatedly been witnessed, often without causing serious floods.

These figures serve to illustrate the extreme variation in maximum rainfall and run-off conditions in different sections of the United States. Until reliable criteria for maximum possible storm rainfall can be established for different regions, engineers will be required to pursue the present tentative system of estimating maximum flood intensities and flood-wave volumes on the basis of available flood records and adding a generous percentage by way of factor of ignorance. Twenty-five years ago when records were meager, the tendency was toward under-estimation. To-day, due to greatly improved records indicating higher values, the tendency is toward over-estimation. This is because engineering conservatism runs toward adhering to the same generous factors of ignorance and applying them to values derived from extraordinary floods, which of themselves approach maximum possible conditions.

The Committee urges caution in the application of air-mass analysis by engineers, especially as regards transposing storm magnitudes from one region to another, without first securing the advice of the experts in this subject in the U. S. Weather Bureau. The Committee senses that there is a danger of applying the air-mass theory indiscriminately. Gross over-estimation, resulting in costly blunders in over-design of structures, may result from the transposition of large-scale, protracted cold fronts with attending heavy precipitation from one water-shed, like that of the Ohio River, to other water-sheds which may not be subject thereto. At a conference held May 25, 1937, with Dr. Willis R. Gregg, Chief, U. S. Weather Bureau, at which the Chief of the River and Flood Division of the Bureau was present, the Committee discussed the foregoing situation and emphasized the importance of providing the Engineering Profession with reliable data regarding the magnitude and frequency with which extraordinary rainfall induced by large Arctic air masses is likely to occur in various parts of the United States. It expressed the opinion that research work in this field should be conducted by the Weather Bureau and not by engineers. The Committee was informed that a request for allocation of funds to the Weather Bureau for air-mass analysis had been ignored some years before; also, that the Weather Bureau had loaned to the Office of the Chief of Engineers, U. S. Army, one of its experts in air-mass analysis to assist in the planning of flood-control works by the Corps of Engineers.

In a letter dated November 13, 1937, Mr. Bernard advised the Committee that a group of specialists had been organized by the Weather Bureau to undertake systematic research work relating to the meteorology of floods. Dr. Gregg, by letter, advised the Committee that while these investigations are primarily designed to meet the requirements of the Army in order to provide a safe basis for hydrologic analysis of spillway and waterway capacities, the results, in due course, would be made available for general use. These investigations are designed to regionalize the country as to storm-producing potentialities and should indicate the limitations governing the transposing of storms.

Historic Floods.—The great destructive floods of the past afford the most reliable index as to what the future may bring. Nothing is more impressive to the lay mind, or more helpful to engineers and to city, State, and regional planners, than to be able to visualize the record of disastrous flood occurrences as far back as historic data permit. The oft-cited fact that the public at large soon forgets the lessons taught by great floods is due largely to such lessons getting lost from view for lack of ready reference to them. The child of to-day still learns in school about the Johnstown flood of 1889 (caused mainly by the breaking of a dam), but other dire flood catastrophes are not brought to its attention, chiefly because teachers are not posted on the subject, much less have opportunity to inform themselves concerning either the frequency or the magnitude of such events. Even the Engineering Profession suffers from this lack of information and tends to become "wedded" to mathematically deduced super-floods derived from stream-flow records which cover, in the majority of cases, periods entirely too short to afford adequate bases on which to predicate large public expenditures for flood control. There lies danger in attaching too much importance to statistical methods applied to such records. For most

rivers of importance, facts concerning destructive floods extend back to the early days of settlement and are not difficult to find if diligent search is made. For several rivers, such historic data have already been compiled with considerable degree of completeness. However approximate some of this information may be, it possesses attributes of reliability that carry an impressive message.

Among the reports issued during the past year (1937) it is gratifying to find several in which mention is made of great floods which antedate the period covered by continuous records. It is to be regretted that in each case lack of space prevented the insertion of more complete information.

In its Progress Report for 1935 the Committee recommended[15] that an inventory be made of the destructive floods of the past, as complete as possible as regards meteorological conditions, synchronism in the arrival of flood crests from tributaries, flood heights noted on permanent structures, and extent of damage inflicted. Such information, published in readable form and not encumbered by references to minor flood occurrences which caused no damage, should take the form of a general reference book on floods for the use of engineers as well as laymen.

Had information of this kind been readily available in the past, it is believed that the great floods in the Ohio River of 1936 and 1937 would have found less valuable property to destroy. It has been known for a quarter of a century that great floods occurred in the Ohio at Pittsburgh, Pa., and at Cincinnati, Ohio, as far back as 1763 and 1774, respectively, which, occurring as they did before the days when Man had begun his encroachment on Nature's flood-plains, were plainly indicative as to what could be counted on to recur some day, greatly aggravated by the presence of civilization. These early floods had been mentioned in print long ago, but not where people interested could readily turn to read about them. When the record floods of 1936 and 1937 occurred, they reached heights totally unexpected, except on the part of a few engineers and meteorologists.

Cloudburst Floods.—There is no phase of flood protection that finds itself so handicapped for lack of data and general understanding as that relating to cloudburst floods. These affect more especially the smaller streams which flow in narrow valleys. Devastation, there, often is complete yet not extensive enough to warrant large expenditures for protection. Several discussions have appeared during the year concerning rates of cloudburst run-off. The number of formulas and graphs submitted evidence the greatly increased mass of information that has become available. The wisdom of platting data pertaining to cloudburst floods on the same diagrams with floods from other causes is questioned. It is felt that floods of different meteorological origins are best treated as subject to separate laws. Noteworthy is the paper by Royce J. Tipton, Assoc. M. Am. Soc. C. E., entitled "Characteristics of Floods in the Southern Rocky Mountain Region," previously mentioned,[14] in which much space is devoted to the causative factors of cloudbursts. *Water Supply Papers 773-E, 796-B* and *796-C*, of the U. S. Geological Survey, describing floods,

[15] *Proceedings*, Am. Soc. C. E., February, 1936, Recommendation No. 2, pp. 204–205.

respectively, in New York State, Republican and Kansas Rivers, and in La Cañada Valley, California, are also of interest in this connection.

As in the case of floods due to other causes, the meteorological aspects of cloudburst occurrence need considerable clarification, especially as regards the important influence of certain topographic features coupled with the presence or absence of trees. The presence of heavy forest cover on mountain slopes appears to act as a deterrent to the formation of local air currents such as are associated with cloudburst phenomena. In previous reports the Committee has urged the need for systematic study of cloudburst floods. It understands that the U. S. Geological Survey, through its office in Salt Lake City, Utah, is engaged on a compilation of data on cloudburst rainfall and run-off. It expresses the hope that, in view of the increasing demand for information, provision may be made for the immediate publication of these data.

The Committee expresses its appreciation of the valuable discussions of its last progress report, which have appeared in the *Proceedings* of the Society.[16] These discussions confirm the Committee's attitude in the matter of providing a central agency to be a clearing house for flood data, and have proved helpful also in other respects.

The Committee submits the following recommendations:

(1) That Congress be urged to designate a suitable Bureau of the Government to act as a permanent clearing house for factual data pertaining to floods, and to publish such data in a form convenient for use in the formulation of flood-control projects as well as for the information of the general public; and, furthermore, that the Bureau so designated be furnished with adequate funds and specially trained personnel for its efficient operation. The Committee suggests giving consideration to either the Water Resources Branch of the U. S. Geological Survey or to the Weather Bureau of the U. S. Department of Agriculture.

(2) That the application of Federal funds be urged for assembling and publishing factual data pertaining to the great historic floods which constitute the background of flood experience in the United States, to the end that this information may be available for the ready reference of engineers, planning commissions, and others entrusted with flood-control problems.

(3) That, until such time as more progress has been made in the study of the meteorology of great floods, engineers engaged in the design of flood-control works exercise caution in applying air-mass theories and in the transposing of great storms from one water-shed to another.

Respectfully submitted:

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Committee on Flood-Protection Data.

December 27, 1937.

[16] Discussion on this report has appeared in *Proceedings*, as follows: September, 1937, by Messrs. John C. Hoyt and H. K. Barrows; October, 1937, by Robert F. Ewald, M. Am. Soc. C. E.; and December, 1937, by C. S. Jarvis, M. Am. Soc. C. E.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

INTERACTION BETWEEN RIB AND SUPER-STRUCTURE IN CONCRETE ARCH BRIDGES

Discussion

By Nathan M. Newmark, Jun. Am. Soc. C. E.

Nathan M. Newmark,[37] Jun. Am. Soc. C. E. (by letter).[37a]—The comments and criticism offered in the discussions of the paper are appreciated. Many of the points raised are of great interest.

Attention is called by Mr. Floris to methods of analysis for the arch with superstructure. The writer purposely did not discuss at any length the matter of analytical procedures. The paper was concerned with the question of why to make analyses rather than how to make them.

Mr. Baron and Professor Cross have stated the main point, namely, that it is necessary to decide what the structure can be made to do, qualitatively, before any detailed quantitative study can profitably be undertaken.

If the rib provides all the necessary flexural resistance to live load, as is usually the case, over-stress in the deck due to participation in the action of the rib may be no more serious than over-stress due to secondary stress in steel trusses. It is certainly better in such a case to detail the structure so as to make the deck flexible, and thereby reduce the participation flexure, than to attempt blindly to analyze the stress in the deck and increase its size and stiffness to provide for indicated over-stress. Such a procedure may make matters worse unless the designer knows qualitatively what the effect of his revisions will be on the structure.

On the other hand, if the superstructure is counted on to provide flexural resistance, over-stress in the deck and columns may be serious.

In certain cases where the controlling relations are relatively clear, it is possible to foretell the action of the structure. The flitch-beam concept enables one to provide a proper division of flexural resistance between rib and deck by choice of their depths.

Note.—The paper by Nathan M. Newmark, Jun. Am. Soc. C. E., was published in September, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: February, 1937, by A. Floris, Esq.; and April, 1937, by Messrs. Frank M. Baron, A. A. Eremin, and Hardy Cross.

[37] Research Asst. Prof. of Civ. Eng., Univ. of Illinois, Urbana, Ill.

[37a] Received by the Secretary January 13, 1938.

The writer agrees with Mr. Eremin that the structures in Fig. 1 are too short to be representative of arch bridges in general. However, the results obtained from analyses of these structures showed essentially the same characteristics as the results of analyses of nine-panel arches similar to those tested by Professor Wilson,[18] influence lines for one of which are shown in Fig. 4. With the shorter structures it was possible to study a greater number of variables. The results were not intended to be used quantitatively in any case. The results of a small number of experiments, whether analytical or laboratory tests, have little statistical value when the number of variables involved is large.

Reference to temperature and deformation strains, instead of stresses, was made purposely. A temperature change or a change in span requires a corresponding change in length of the elements of the structure, and causes a distortion of the arch rib. This distortion must take place, although it can be localized, or it can be distributed by the details of the structure. The stiffer the structure and the higher the modulus of elasticity of the material, the greater will be the stresses corresponding to the more or less fixed deformations. Referring to temperature strains instead of stresses, serves to keep this point in mind. Moreover, the temperature strain is particularly significant in any consideration of ultimate strength of the structure. When combined with high stresses due to loading, the additional strain due to temperature effects and other fixed deformations accounts for only a small increase in stress on account of the nature of the stress-strain curve for concrete.

It does not seem necessary to enter into a detailed presentation of the implications of the flitch-beam concept regarding the effect of relative depth on relative stress of two members so connected as to have the same deflection curve. If the floor and the rib of an arch bridge are connected by closely spaced flexible columns, it is evident that they will have the same deflection curves, and consequently the same angular changes per unit length along the horizontal. Therefore they will have unit flexural strains, and consequently flexural stresses, roughly in proportion to their depths. If the columns are flexible but not very closely spaced, the relation is still approximately true for loads at the column points. When the columns are not flexible, there may be considerable local flexure of the more flexible member, whether it be rib or floor. Obviously, a simplified approximate relation has limitations. However, that does not make it useless.

The interaction stress in the floor is that due to the deflection of the arch rib. The writer's statement: "The stress due to interaction flexure in the girder is to the flexural stress in the rib approximately as the depth of the girder is to the depth of the rib," is an attempt to interpret in a simple manner the behavior of the structure. The less stiff the columns, and the shorter the panel length, the more nearly true will the approximation be. The rise ratio of the arch is not involved in the development of this relation.

[18] *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 1486, Fig. 41(*a*).

The Landquart Bridge at Klosters, Switzerland, referred to by Mr. Eremin, was mentioned in the paper as an example of the structure consisting of a flexible rib with stiffening girder supported by flexible columns. Practically all the flexural resistance in this case is centered in the floor. The arch rib is polygonal in shape and carries the dead load thrust. The rib has actually participation flexural stresses approximately one-fourth the interaction flexural stress in the floor since the depth of the rib is about 10 in. and that of the floor is about 3 ft.

Mr. Eremin has stated some of the variables on which the interaction stresses depend. One might add other variables having to deal with: (1) The sources of stress (including the dead load-live load ratio, lateral forces, shrinkage, and temperature); (2) sizes of the members; (3) shape of the structure; (4) properties of the material, and variations in properties; and (5) conditions of support. It is evident that the problem is not a simple one. For this reason, the writer attempted to develop simplified "pictures" of the action of the structure rather than to indicate how an exact analysis might be made or to report statistical results of analyses or tests. Professor Cross has indicated how such "pictures" of structural action may be useful.

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DISCUSSIONS

FLOW CHARACTERISTICS IN ELBOW DRAFT-TUBES

Discussion

BY C. A. MOCKMORE, M. AM. SOC. C. E.

C. A. MOCKMORE,[30] M. AM. SOC. C. E. (by letter).[30a]—There is ample reason to agree with Professor Mavis that "the investigation of flow in draft-tubes has little more than made its 'debut' in the hydraulic laboratory." The findings made in this study and the suggestions made for the design of draft-tubes are not regarded as final. In this day of research, even the validity of the Newton law of gravitation is subjected to careful scrutiny.

Mr. Fee refers to Fig. 1 and the discussion following Equation (5), and states that he "would expect a high, not a low, pressure at Point *d* in conformity with the usual rule that high pressures go with low velocities." This belief is apparently based on the Bernoulli theorem, and would be correct for a single particle of water, but in Fig. 1 two different particles are under consideration.

Consider a case of two particles of water such as at Points *b* and *c* of Fig. 1 if they are moved up stream a reasonably short distance, say, 1 ft. Would the two particles be subjected to pressures of equal magnitude? Certainly the particle near the center of the pipe would have a greater velocity head than the one near the walls of the pipe. If the two particles possessed the same total energy the pressure near the walls would be greater than at the center of the pipe. This seems questionable. If there is a difference it could not well be detected by the pressure orifice of the Pitot tube used in these experiments. If the pressures are practically uniform across the pipe section and the filamental velocities are highest near the center of the pipe then the highest total energy content would be near the center of the pipe, as indicated for Section 1 of Bend No. 1, Fig. 7.

NOTE.—The paper by C. A. Mockmore, M. Am. Soc. C. E., was published in February, 1937, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: April, 1937, by F. T. Mavis, M. Am. Soc. C. E.; May, 1937, by Jerome Fee, Assoc. M. Am. Soc. C. E.; June, 1937, by Messrs. R. E. B. Sharp, and L. F. Harza; September, 1937, by Ellery D. Fosdick, Esq.; and October, 1937, by Howard L. Cooper, Esq.

[30] Prof. and Head, Civ. Eng. Dept., Oregon State Agri. Coll., Corvallis, Ore.

[30a] Received by the Secretary January 3, 1938.

Now consider particles at Points *b* and *d* of Fig. 1 again, just before they start around the pipe bend. Suppose each particle possesses approximately the same pressure head, but different velocity heads. Then an instant later each particle starts its course around the bend. Particle *b* will have the greater centrifugal force and thus will tend to crowd Particle *d* toward the center of curvature. Two fundamental conditions were necessary to induce the spiral, namely, the presence of a curve in the channel to induce centrifugal force, and a difference between filamental velocities near the center and walls of the conduit.

Mr. Sharp thinks that the writer is not justified in submitting general recommendations for the design of elbow draft-tubes on the basis of the tests submitted in the paper. Of course, the recommendations are not drawn from the tests described only, but from other sources, such as a review of the literature, inspection of turbines in operation and repair, and from visits to various turbine laboratories where scores of model draft-tubes of almost every conceivable shape have been tested. For example, at one commercial laboratory visited in 1932, a test was described in which a movable false bottom was installed in an elbow tube such as one might imagine for Tube No. 2, Fig. 12, between Sections 5 and 10, so that the vertical dimension along Section 7 (termed the Δ-distance in this discussion) might be reduced. In the test referred to, the tube efficiency was increased by reducing the Δ-distance.

The data on the areas at various sections of Draft-Tube No. 2 are given in Table 2, and are compared to a 6° conical tube.

Mr. Sharp states that tests on model draft-tubes should be used as a guide for tests with model turbines "but should not be used as the bases for the construction of large tubes in the field." It would be difficult to convince many engineers interested in model research that such work had no value in relation to the design and construction of large water-turbine developments. Very little experimental data can be had on such large developments as that on the Columbia River. Changes are difficult and expensive to make, and measurement of flow of water through the turbines is especially troublesome and questionable in accuracy.

The trumpet-shaped vertical leg of the draft-tube need not extend far from the exit of the turbine runner, so that the allowable values of flare are not exceeded, somewhat as an easement spiral is placed on a railroad curve. Suppose the efficiency of Draft-Tube No. 2 were higher than that of Tube No. 4, as Mr. Sharp points out, one wonders why he did not show experimental evidence or other proof, that it could have been due to the Prasil vertical leg. Experiments of Hoffmann (8),[3] Gibson and Labrow (30),[3] and Andres (31),[3] indicate that the trumpet-shaped tube has as high, if not higher, efficiency as a straight cone. As indicated in Figs. 18 and 19, there was greater dead-water space in Tube No. 4 than in Tube No. 2. Surely this should not be ascribed to the Prasil vertical leg. Would it not more logically be due to too great a Δ-distance? In this same connection

[3] Numerals in parentheses, thus (8), refer to corresponding numbers in Appendix I of the paper, *Proceedings* Am. Soc. C. E., February, 1937, p. 283.

Mr. Harza suggests that considerably more improvement would result from a still shallower and wider section of a pipe bend, which would call for additional lessening of the Δ-distance of an elbow tube.

The loss in flow through the experimental pipe bends appears to have interested, not only Mr. Sharp and Mr. Harza, but brought forth the description of an interesting experiment by Mr. Fosdick. His special Bend No. 2 gave the most favorable loss factor, and, in general, seems to verify the writer's suggestion, (7(*c*)), for the design of the bend of an elbow draft-tube.

It is regretted that Mr. Cooper did not submit the design of the Draft-Tube No. 5, which showed such fine characteristics in operation. He states that it "was designed so that the entire whirling component was transferred into velocity head." The test would really be met when this whirling component is transferred into effective head on the turbine.

Mr. Cooper submits Table 8, as evidence that the splitter should extend completely around the bend as in Draft-Tube No. 2, ostensibly to refute Conclusion (10), and then proceeds to give his own suggestions for design to include "one horizontal splitter which does not extend more than one-third of the way around the elbow." No reason is given for the apparent inconsistency.

The writer earnestly wishes to extend appreciation to all those who have helped make this paper possible, and particularly to those who have given of their time and effort in criticising the manuscript and submitting discussions.

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DISCUSSIONS

MEASUREMENT OF DÉBRIS-LADEN STREAM FLOW WITH CRITICAL-DEPTH FLUMES

Discussion

BY MESSRS. R. L. PARSHALL, AND MARTIN A. MASON

R. L. PARSHALL,[11] Assoc. M. Am. Soc. C. E. (by letter).[11a]—The authors have developed an interesting measuring device for the special purpose of measuring flood flow in steep channels where the streams carry heavy bed-loads and high percentages of suspended material.

The name applied to this new measuring flume, perhaps, is somewhat misleading. The flow passes through the section of critical depth at a point approximating the up-stream ends of the parallel walls of the structure. The section, or point of critical depth, varies with the rate of discharge and, as stated by the authors, is not at a definite place. The gage section, as chosen, is considerably down stream from the point of critical depth, and, therefore, the observed values of the H_2 head are always less than the critical. It is probable that a better index point for determining the head would be at a point slightly up stream from the upper end of the parallel section where greater depths would exist. For larger values of the head, thus observed, the error in depth measurement then would be less effective in the final computed discharge. This point no doubt has been investigated.

The authors have studied the accuracy of the Parshall measuring flume for their special purposes and report that if the floor of the converging section is made level and the length of this section of the structure is increased, little or no effect will appear in the discharge characteristic of the flume. However, when the floor is sloped to 5% in the direction of flow, the indicated discharge is less than the actual flow. This would be an obvious result because the control is moved up stream from the throat to a point at or near the upper end of the converging section. Moreover, the depth at the standard H_a point would be less because of the higher velocity of the water passing this section. This alteration materially changes the characteristics of flow, and the law of dis-

NOTE.—The paper by Messrs. H. G. Wilm, John S. Cotton, and H. C. Storey, was published in September, 1937, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[11] Senior Irrig. Engr., Bureau of Agricultural Eng., U. S. Dept. of Agriculture, Fort Collins, Colo.

[11a] Received by the Secretary January 10, 1938.

charge for the standard setting is not applicable, particularly for the shallower depths of flow.

Sufficient data are not presented by the authors in their discussion of results of testing the Parshall measuring flume, under modified settings, to permit complete analysis of their conclusions, "as shown in Tables 1 to 7." The writer finds only one table dealing with the modified flume where the floor in the converging section has been sloped downward. The discharge formula, Equation (3), as stated for the flow through the 6-in. flume, is apparently not for a standard setting. At the hydraulic laboratory at Fort Collins, Colo., this particular size of flume was calibrated where the law of discharge was found to be:

$$Q = 2.06\, H_a^{1.58} \qquad (9)$$

based on twenty-three observations for discharges ranging from 0.05 to 2.42 cu ft per sec. In this series there were two tests in which the deviation approached 6 per cent. In one case the H_a head was only 0.093 ft, and for this shallow depth it is believed the control at the throat was ineffective; however, the actual difference between the observed and computed discharges was only 0.003 cu ft per sec. For practical purposes this difference is not significant. For the other test in question, the H_a head was 0.217 ft, with a difference in the observed and computed discharges of 0.01 cu ft per sec. Another test in this series with the H_a head at 0.204 ft indicated a difference of only 0.001 cu ft per sec. These slight variations in discharge, for low heads, are thought to be due to the approaching limit of control at the throat for shallow depths. For the remaining twenty tests the mean deviation between the observed and computed discharges was 0.8 per cent.

An inspection of the data in Table 1 discloses that the 3% slope of the floor of the converging section, in the direction of flow, materially affects the indicated rate of discharge through the flume when based on a standard setting. For the very limited data presented concerning the four tests on the 1-ft Parshall measuring flume, having a sloping floor and carrying a débris-laden flow, the following relation has been found to exist: The gage point, H_a, is practically 3 ft up stream from the crest. At a 3% slope this point is 0.089 ft higher than the crest. When this increment of 0.089 is added to the observed H_a head the standard discharge for these corrected heads agrees reasonably well with the computed discharge shown in Table 1 as based on the San Dimas calibrations.

The development of a measuring device to meet extreme adverse conditions of heavy bed-load carried in the stream, has not yet been fully accomplished. The authors have intimated strongly that the Parshall measuring flume, when operated under the field conditions imposed on the experimental San Dimas Forest area, has failed to register the exact rate of flood-flow discharge. Experience in the field with weirs of various types, as well as the ordinary rating flume, has demonstrated that neither of these devices could be used successfully to measure flood flows such as were encountered in this experimental forest area. There were experimental data, both in the laboratory and in the field, in evidence prior to the recommendation of this type of measuring device for the San Dimas area, which indicated that the Parshall measuring flume did possess

characteristics favorable in meeting such adverse conditions of measuring débris-laden stream flow. In the Fort Collins Laboratory numerous observations have been made to ascertain the effect of sand and gravel on the indicated rate of discharge where such débris was carried to, and passed through, the flume. No material effect was noted in the indicated rate of flow because of the fact that any bed-load brought by velocity of approach to the upper end of the structure was readily moved on through it, owing to the convergence of the flow and the increasing velocity through that part of the structure in which the control and measurement of the discharge was effected.

FIG. 14.—CONDITION OF FLOW THROUGH 6-FOOT PARSHALL MEASURING FLUME AT EAST PORTAL OF LARAMIE-POUDRE TUNNEL WHEN MEASURED AND COMPUTED DISCHARGES WERE IN REASONABLE AGREEMENT.

For a number of years, prior to 1937, at the east portal of the Laramie-Poudre Tunnel, 60 miles west of Fort Collins, a 6-ft Parshall flume was operated to measure the flow through this tunnel. The flume operated under adverse conditions as follows: First, the stream from the portal of this tunnel was abruptly turned 90° into a relatively short channel in which the measuring flume was placed; and, second, the excessive quantity of heavy material carried out of the tunnel was moved down and through the flume. Numerous current-meter check measurements had been made of the discharge by official hydrographers and others, which showed the rate of flow as indicated by the flume to be within reasonable approaches of accuracy. In Fig. 14 is shown the condition of flow through this 6-ft measuring flume on July 2, 1932. On July 10, a current meter gaging of the discharge was made of essentially the same flow and degree of gravel deposit, with the thread of the stream flowing along one side of the channel. For this gaging, the mean H_a gage height was found to be 1.18 ft, the computed discharge 31.25 cu ft per sec, and the metered discharge, 32.6 cu ft per sec, with a resulting deviation of about 4 per cent. Under such adverse conditions extreme accuracy cannot be expected.

The Parshall measuring flume has been accepted throughout the irrigated Western States as a practical and reliable means of gaging the rate of flow of water diverted from streams and reservoirs under officially decreed rights. It is estimated that there are about 2 500 flumes in operation in Colorado at this time. This measuring flume is widespread in its use and is serving a useful purpose in this, and many foreign, countries for measuring both irrigation and power supplies, in sanitary engineering, and in industrial plants.

The first practical installation of this type of measuring device was a 10-ft framed structure built in the Las Animas Consolidated Ditch, near Las Animas, Colo., in 1926. The successful operation of this first flume resulted in the construction of other larger flumes in the Arkansas Valley in Colorado. These flumes were of reinforced concrete, and ranged in size from 15 to 40-ft throat widths. Prior to the construction of the 40-ft reinforced concrete flume in the Fort Lyon Canal, at La Junta, Colo., the discharge formula,

$$Q = 150\, H_a^{1.6} \qquad (10)$$

was set up for the purpose of designing this structure. After this flume was put into operation (November, 1928), current meter gagings of the discharge were made covering a period of several months. These gagings were made principally by the official State Hydrographer; however, some of the observations were made by the writer and others. This series of meter measurements, which consisted of twenty-three gagings, was found by comparison with the discharge, computed by Equation (10), to be as follows: Twelve indicated a plus deviation and eleven, a minus deviation; for nine of these comparisons, the difference was less than 1% and the maximum deviation for any one of the twenty-three observations was 2.5 per cent. For all flows measured, the submergence was well below the limit of interference. These check measurements of the discharge ranged from about 130 to 1 460 cu ft per sec. A very large number of current meter check measurements of the discharge through other Parshall measuring flumes of various sizes show close agreement between the observed and computed rate of flow.

The authors' report of tests on the newly developed critical-depth measuring flume, under laboratory settings, where the range of flow varied from less than 1 cu ft per sec to more than 50 cu ft per sec, shows marked consistency in the accurate measurement of flow. The practical operation of the flume in the field, for both large and small sizes, however, may indicate some shortcomings, especially where it is required to measure a flow of a few cubic feet per second through a structure having a capacity of more than 1 000 cu ft per sec. Experience with the Parshall measuring flume on the San Dimas Experimental Forest appears to indicate that better results would be obtained by having greater depths and less width of throat.

The writer is much interested in the development of a practical measuring device for the special purpose of gaging flood flows which carry an excessive quantity of bed-load. The authors' investigation of such a new device, to meet these requirements, is encouraging when viewed from the standpoint of laboratory results. There appears evidence, however, that from the standpoint of

field experience this new critical-flow measuring flume has not been entirely successful.

Corrections for *Transactions:* In Columns (1) of Tables 1, 2, and 3, of the paper change the heading to read "Observed Head at Stillwell Inlet, in Feet"; to the side heading "High-Velocity Flumes (San Dimas Design)," add "Rectangular Cross-Section, Level Flume Floor"; in the sub-titles to Table 2 change Equations (9*a*), (9*b*), and (9*c*) to read Equations (7*a*), (7*b*), and (7*c*), respectively; change Equation (6) to read,

$$``Q = \frac{1.120\, B^{0.040}}{H_n}\, B\, H^{1.5} \qquad (6)$$

in which $n = 0.179\, B^{0.320}$"; in the sub-titles to Table 3 change Equations (10*a*), (10*b*), and (10*c*) to read Equations (8*a*), (8*b*), and (8*c*), respectively; change word, "Portion," in caption to Fig. 2, to read "Position"; and change words, "Control-Depth" in caption to Fig. 6 to read "Critical Depth."

MARTIN A. MASON,[12] JUN. AM. SOC. C. E. (by letter).[12a]—The accurate measurement of the discharge of streams carrying silt and solid materials has great interest for the hydraulic engineers of Europe, particularly those in Switzerland and the French Alps. Most of the streams in that section are of the nature of mountain torrents, transporting considerable solid material, and are usually characterized by very turbulent flow at high velocity. Because of the intensive power development of the streams the accurate measurement of their discharges is of more than ordinary importance. More or less satisfactory discharge determinations are generally made by the use of current meters at either a natural or an artificial control section; the use of weirs, Parshall flumes, Venturi flumes, and similar devices being precluded, except in unusual cases, on account of their lack of reliability when measuring silt-laden flows. The need for a simple, reliable, and accurate means which may replace current meter methods in the measurement of such flows is recognized; and it is to be hoped that the complete San Dimas investigations will represent, at least, a first step toward the development of such a method.

A particularly difficult problem in silt-laden flow measurement was recently presented by the need of gaging the La Sésia torrent, a small mountain stream near Le Sautet Dam in the Préalpes of France. The normal flow of about 2 m^3 per sec (70.63 cu ft per sec), with velocities above the critical, carries solid materials, ranging in size from fine sand to boulders 30 cm or 35 cm (12 in. to 14 in.) in diameter, in quantities varying from 10% to 20%, by volume, of the discharge. In times of flood or melting snow this value may become as high as 50 per cent. If possible, it was desired to establish a gaging flume in the stream which would permit a reliable, accurate determination of the discharge under varying flow régimes.

The lack of information on the performance of such devices under these conditions led to co-operative studies being made at the hydraulic laboratories of the School for Hydraulic Engineers and of Ateliers Neyret-Beylier and

[12] Scientific Aid and Junior Mech. Engr., National Bureau of Standards, Washington, D. C.; Freeman Scholar, 1937, Grenoble, France.

[12a] Received by the Secretary January 27, 1938.

Piccard-Pictet, at Grenoble, France, with the purpose of developing a Venturi or a Parshall flume which would function satisfactorily under the unfavorable conditions obtaining in the La Sésia.

These studies did not succeed in the development of a flume of either type which was entirely suitable. It was found at once that the Parshall flume was practically valueless due to the deposition of some of the transported solids, particularly the larger stones and boulders, wherever shooting velocities did not exist. It is interesting to note that exactly the same result was obtained at San Dimas in the tests of the altered Parshall flumes with débris-laden flow. A rectangular flume with converging side walls, designed on the principle that the super-critical velocities obtained would be sufficiently high to carry all the transported solids through the measuring section without deposition, was also unsatisfactory. It was found impossible to attain velocities high enough to keep the flume clean at all stages of flow.

The solution finally arrived at, although it is admittedly crude and susceptible of improvement, is nevertheless of interest for its ingenuity and entirely different approach to the problem from that adopted at San Dimas. A rectangular wooden flume about 13 m long, 3 m wide, and 1.5 m deep (42.7 ft by 9.8 ft by 4.9 ft), was substituted for the natural stream channel in a short section where the slope was relatively small (2 cm per m or 0.24 in. per ft). The stream was led into and away from the flume by rock-bundle transitions. A movable gate was installed at the down-stream end of the flume.

The technique used in making a gaging is as follows: With the flume gate normally open and the stream flowing freely through the flume a deposit of the larger solids carried is formed due to the reduction in velocity in the flume brought about by the reduced slope of the bed. When it is desired to make a measurement of normal flow (that is, no flood conditions) the flume gate is partly lowered, and the increased velocity under the gate scours out the deposited material. With the velocities existing in normal flow this operation requires about 10 min. When the flume is clean, as determined by sounding-rod observations, the gate is completely closed and the measurement begun. Velocity measurements are taken with current meters at a number of points dependent upon the conditions of flow. No trouble is experienced with deposits forming in the flume during the short time required to make the meter measurements because, with the gate closed, there is a reduction of velocity in the flume causing, in turn, a jump to form some distance up stream. The solid material, with the exception of the finer sand, is thus deposited in the stream bed in the vicinity of the jump, only the now relatively clear water passing through the flume and over the gate. With the reduced velocity obtained and with the measuring section located about 10 m (32.8 ft) down stream of the flume entrance the flow conditions are very favorable to accurate velocity measurements. Upon completion of the gaging the gate is opened, and again free flow through the flume, at a velocity above the critical, takes place. The increased velocity in the stream bed above the flume entrance scours the deposit formed there by the hydraulic jump during the gaging, leaving only the largest of the boulders that have lodged there. These are removed manually from time to time as required.

In times of flood or of melting snow, when the solid content of the stream may become as high as 40 to 50%, by volume, the gaging is made with the flume gate open, correction being made for the deposit in the flume as determined by sounding-rod observations. The error in the measurement of the flow at such times is of the order of 10%, whereas for normal flow conditions the error does not exceed about 5 per cent.

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DISCUSSIONS

WATER TRANSPORTATION VERSUS RAIL TRANSPORTATION

A SYMPOSIUM

Discussion

By J. E. Goodrich, Esq.

J. E. Goodrich,[21] Esq. (by letter).[21a]—Much information of interest is contained in the papers of this Symposium. Fully realizing his incapacity to present any meritorious critical discussion of the opinions of experts in the matter, the writer requests that his comments be considered in the nature of inquiry.

Major Putnam has defined one important characteristic of the movement of commerce on an inland waterway—that is, the load factor of the floating equipment engaged therein. It is to be hoped that, in his closing discussion, he will present data as to actual existing load factors based on his definition. It may be found that they are quite low. For instance, he indicates that the commercial fleet in active operation in 1935 on the waters under consideration in his paper, included towboats with an approximate horse-power of 176 000 and barges with a carrying capacity of 1 600 000 tons and, furthermore, under his comparison of costs, that the annual traffic to be handled is about 5 500 million ton-miles. Using these data, it appears that the actual average load factor of the towboats is about 24% and of the barges about 8.7 per cent. It is possible that the foregoing is an improper derivation, or that after the waterways have been fully developed, the average operating load factor may be somewhat improved. At any rate, before a reliable comparison between water and rail costs can be made, a reasonable determination of the probable load factor is essential.

It appears that the main purpose of comparisons between the costs of transportation by land and inland waterways is to determine whether additional

Note.—The Symposium on Water Transportation Versus Rail Transportation was presented at the meeting of the Waterways Division, Little Rock, Ark., April 25, 1936, and published in September, 1937, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: October, 1937, by George Hartley, Esq.; December, 1937, by W. D. Faucette and J. E. Willoughby, Members, Am. Soc. C. E.; and January, 1938, by C. D. Bordelon, Esq.

[21] Senior Engr., U. S. Engr. Office, Upper Mississippi Val. Div., St. Louis, Mo.

[21a] Received by the Secretary January 3, 1938.

waterway improvements should be undertaken. Obviously, an investigation for that purpose should take into account the extent to which existing transportation facilities are adequate. Reduced to a simple case in which a waterway under consideration for improvement is to serve an area now adequately served by rail, should not the comparison be based on the total cost of transportation in the area served by the two systems before and after the waterway is improved and placed in service? Before the waterway is put into service the costs include the fixed and operating charges of the railroad alone. When the waterway is in service the total cost includes the fixed and operating charges of the waterway and the carriers thereon plus the same fixed charges on the railroad as existed previously, as well as operating charges on such freight as is left to the railroad. The only way the railroads could avoid these fixed charges (which persumably the public must pay) would be to abandon the service. Even if this were permissible, it would entail a destruction of capital which, "in the long run," would be paid for by the public. If this view is entitled to consideration, it would appear necessary to include in the balance sheet of water *versus* rail transportation an evaluation of the public loss due to rendering the existing rail facilities less economically efficient.

It may be contended that by reducing the cost of certain commercial movements, water facilities would stimulate industry so that the traffic diverted from the rail line would be restored from other sources. Industry, however, is affected by taxes as well as by direct transportation costs. If the waterway improvement increases the tax bill to a greater extent than it reduces the direct transportation bill, it is difficult to foresee a resulting general stimulation of industry so as to bring about increased commerce in the country as a whole. On the other hand, a waterway which results in a substantial reduction in combined transportation and tax costs may be an actual benefit to the railroads as well as to other interests. In investigating the merits of a waterway to serve an area already adequately served, it should not be assumed that benefits of this character will arise until it has been shown from other considerations that the waterway is warranted and will actually reduce total costs. It should be shown that the horse exists before it is assumed that the cart will be drawn. Thereafter, these factors, too, should be taken into account and thoroughly analyzed in determining the extent to which the waterway improvements are worth while.

A knowledge of relative cost without consideration of the relative value of the service obtained is not sufficient to indicate the worth of an investment. It is generally recognized that waterway and rail transportation are not equally expeditious, flexible, and dependable, and these inequalities should enter into the economic study. Perhaps the best indications available at this time of shippers' views as to these inequalities are the differentials between rail and barge line rates necessary to place the two types of service on a competitive basis where common carriers are involved. It is possible that shippers' views in this respect may be modified as the waterways become more adequately improved and their services better established.

Economic comparisons of rail and waterway transportation costs are frequently made in which the interest on the cost of the waterway is computed

at a lower rate than that applicable to railroads. Such methods seem improper if the purpose is to show the inherent superiority of waterways aside from the advantage that they possess due to low-cost Federal financing, but proper if the purpose of the comparison is confined to determining the most economical medium under actual present methods of securing capital.

If economic comparisons were limited to reductions in the cost of freight movements, and the foregoing ideas were given application, some proposed inland waterway improvements might not be found so meritorious as to warrant the degree of enthusiasm often afforded them. However, waterways may be beneficial to a substantial volume of pleasure-boat navigation. In some degree waterway improvements, through increase in low stream flows, or the concentration of head, may facilitate the production of power. They may result in flood-control benefits by the storage of flood water, stabilization of banks, improvement of the hydraulic characteristics of streams at flood stages, and the supply of vessels for rescue work, although these benefits may be offset somewhat by the stimulation of objectionable and inadequately controlled concentration of industry in the flood-plains of streams. The availability, in operating condition, of ordinarily excess transportation facilities to meet unanticipated emergencies, is apparently of some benefit. Under certain conditions the undertaking of waterway improvements which approach the border line of economic feasibility may be warranted because it will afford productive employment of labor which, otherwise, would receive non-productive relief funds. Without involving sentimentalism, these items may properly be given consideration in an economic comparison of rail and waterway transportation.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

PRACTICAL APPLICATION OF SOIL MECHANICS

A SYMPOSIUM

Discussion

BY MESSRS. CHARLES SENOUR, DONALD M. BURMISTER, AND DONALD W. TAYLOR

CHARLES SENOUR,[64] ESQ. (by letter).[64a]—In view of the interest in levee construction engendered by the fairly recent floods of New England and of the Ohio River Basin, Mr. Buchanan's paper is most timely. The latter flood (January, 1937), after reaching the Mississippi River, was carried to the Gulf between levees, and one aftermath of its successful passage has been a number of requests for information regarding standard practice in levee design and construction on the Lower Mississippi. This has been given gladly, of course, but with occasional misgivings as to its applicability, particularly in the department of design, to the problems of distant flood plains. The Mississippi River levee is the product of years of experience with the specialized problems of the lower alluvial valley. (Fig. 2 depicts its metamorphosis from the simple trapezoidal cross-section of 1882 through the several types of banquette section to the more satisfactory trapezoidal section of the present day.) There is one school of thought, and a rather large one, that holds to the view that the conditions surrounding levee construction make levee design unavoidably an empirical matter. Experience is invaluable, of course, but unless it is supplemented with a knowledge of the underlying principles that govern soil behavior, experience is distinctly at a disadvantage when it becomes necessary to design in a field not covered by prior operations. It is this inherent shortcoming that the paper by Mr. Buchanan seems particularly helpful in overcoming.

Until the comparatively recent past, there has been little to draw upon except experience. The science of soil mechanics is itself quite young, and its

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[64a] Received by the Secretary October 7, 1937.

application to the field of levee design has lagged considerably behind its application to the related but, in many respects, dissimilar problems of earth dams. A levee is an earth dam, of course, but it is a very specialized form and the circumstances normally surrounding its construction are far less favorable to precision in design than are those which attend the building of a dam.

The main levees of the Lower Mississippi River are as a rule several hundred miles in length, and are tied into high ground (that is, they have an abutment) only at the up-stream end, the basin being left open at the lower end for drainage. The embankment skirts the river edge of a flood plain whose width may be 50 or 60 miles. This plain is composed of alluvium to great depths. Much of it has been worked over by the river, whose migrating meanders, cut-offs, and bank-caving and bank-building operations have produced a geologic hodgepodge of which the ingredients are clay, sand, and silt, with some vegetable matter. These ingredients occur singly and in an endless variety of combinations which follow no pattern and apparently recognize no law except that of change.

It is evident that one cannot very well haul upland earth to build his levee of homogeneous clay; nor can he afford to haul laterally from the possibly better soils of the flood-plain's distant hinterland. If he did so, furthermore, he still would have to found the levee upon the heterogeneous mass of which the area adjacent to the river is composed. The problem, therefore, is to build from the materials immediately at hand, and since these vary so erratically as to make selection seem rather impractical in a majority of cases, it has appeared advantageous to procure the material as close to the embankment as safety permits, so as to shorten the haul. The borrow-pit is usually on the river side, and is quite apt to encounter rather pervious materials after the surface strata are removed. Since these porous materials will be exposed to full hydrostatic head once they are uncovered, it is inadvisable to expose them very close to the embankment, and it is, of course, desirable to utilize as small a proportion of the more pervious materials in the construction as circumstances permit. Deep excavation close to embankment also invites foundation troubles. From the practical standpoint, furthermore, it is not safe to count on going very deep because of moisture.

With these conditions laid down, the best solution has seemed to be to construct from shallow side borrow-pits an embankment of sufficient bulk to compensate for any shortcomings with respect to homogeneity and compaction. When teams were the instruments of levee construction, compaction was automatic, but the size and quantity of construction rapidly expanded beyond the ability of teams to handle alone, and much of the system was completed or under way prior to the comparatively recent introduction of trucks and tractors as important tools. Much yardage has been placed by draglines equipped with buckets of capacities of 3 to 8 cu yd, and tower machines handling buckets of from 10 to 15 cu yd. Machinery of this type (largely inspired by the needs of levee construction) handles earth very efficiently within the limit of its reach, which for the tower machine, incidentally, is about 1 000 ft. Neither by the tower machine nor by the dragline is any deliberate attempt made to compact the fill.

No effort is made to control moisture content, in the sense of attaining the optimum. The susceptibility to wide seasonal variations in moisture of pits adjacent to a river whose range in stage is about 50 ft and the necessity for utilizing the working season to the best advantage had made precise moisture control appear unattainable. A limiting steepness of construction slopes (1 on 2.8) is often prescribed by the specification, especially when much clay is present. The contractor, however, is permitted to use fairly damp earth, subject to his own risk (and subject to warnings from the inspector when he appears to be taking chances). If a slide occurs from the use of material that is too wet, the contractor is required to cut out and remove the embankment to and beyond the sliding surface and to restore it with satisfactory material —all at his own expense. Since this is quite costly, it operates as a deterrent to the use of earth that is dangerously damp.

When the levee is built with hauling equipment it is constructed in layers, with an allowable thickness for vehicles hauling loads of 5 cu yd or more, of 5 ft, and for equipment hauling loads of less than 5 cu yd, 3 ft. When draglines (boom machines) or tower excavators are used, layer construction is not required. In all cases the contractor is required to build to a gross cross-section from 8% to 25% higher vertically at all points than the net cross-section, for which latter alone he receives payment. This surplus is designed primarily to compensate for the lack of compaction, the larger shrinkage allowances pertaining to those methods of construction which involve the least compaction. The shrinkage allowance seeks to insure a levee permanently to the grade desired even with some consolidation of foundation. It is, therefore, liberal.

A series of shrinkage observations made upon levees constructed in the Lower Mississippi Valley has been plotted in Fig. 64, in which the volume of the embankment of various ages is expressed in terms of the volume of the borrow-pit from which it was constructed. Each curve represents a series of experiments on a single unit involving from 8 000 to 100 000 cu yd, except that labeled "New Orleans" which represents the unweighted mean of a group of six experiments on dragline-built units ranging from 160 000 to 660 000 cu yd in volume. In this latter case the results varied so little that individual plotting appeared confusing (from 1.05 to 1.15 borrow-pit volume at the time of completion, and from 1.08 to 0.998 at the end of 30 months).

The curves indicate a pronounced superiority in density of the tractor-built units over that of the dragline-built units, the tower excavators being represented in both groups. One method of construction with tower excavators is to "base in" the fill with one machine and mount a second, following machine upon the material so placed, to complete the cross-section. When this is done, the influence of the tremendous weight of the equipment, supplemented by its vibrating effect, apparently tends (in some soils, at least) to attain tractor densities in the fill.

On a big levee, tractors, draglines, and tower machines all may be engaged concurrently in placing different parts of the cross-section. The work is awarded by contract to the lowest responsible bidder, and there is no method of forecasting before the bids are opened who he may be or what method he will choose to adopt. Furthermore, the vagaries of weather and river stage

may force him to modify his original plan from time to time, and each time he does so, the composition and density of the fill are changed somewhat.

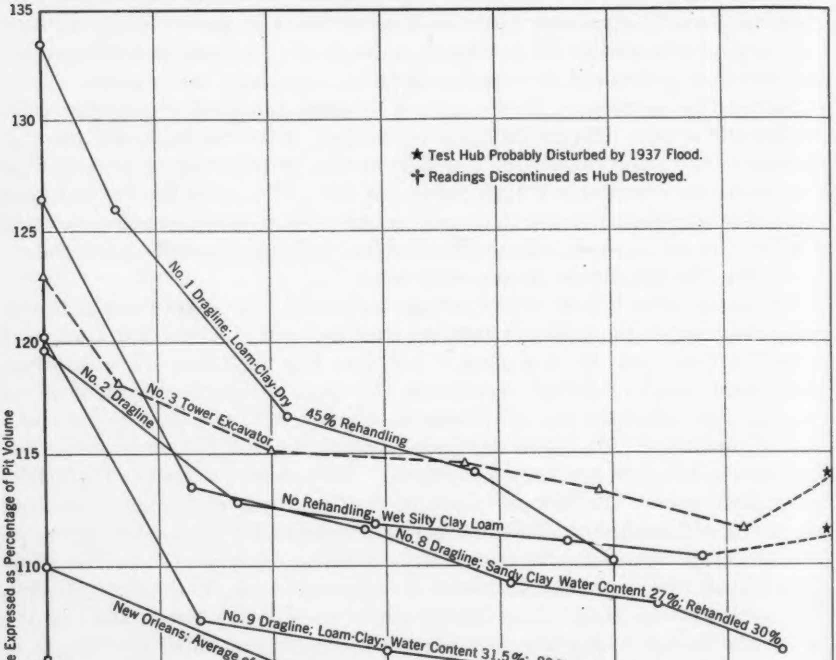

FIG. 64.—SHRINKAGE OF LEVEES

Fig. 65 shows the theoretical shape of a standard borrow-pit. The depth depicted is the limiting value allowed by the specifications and the width,

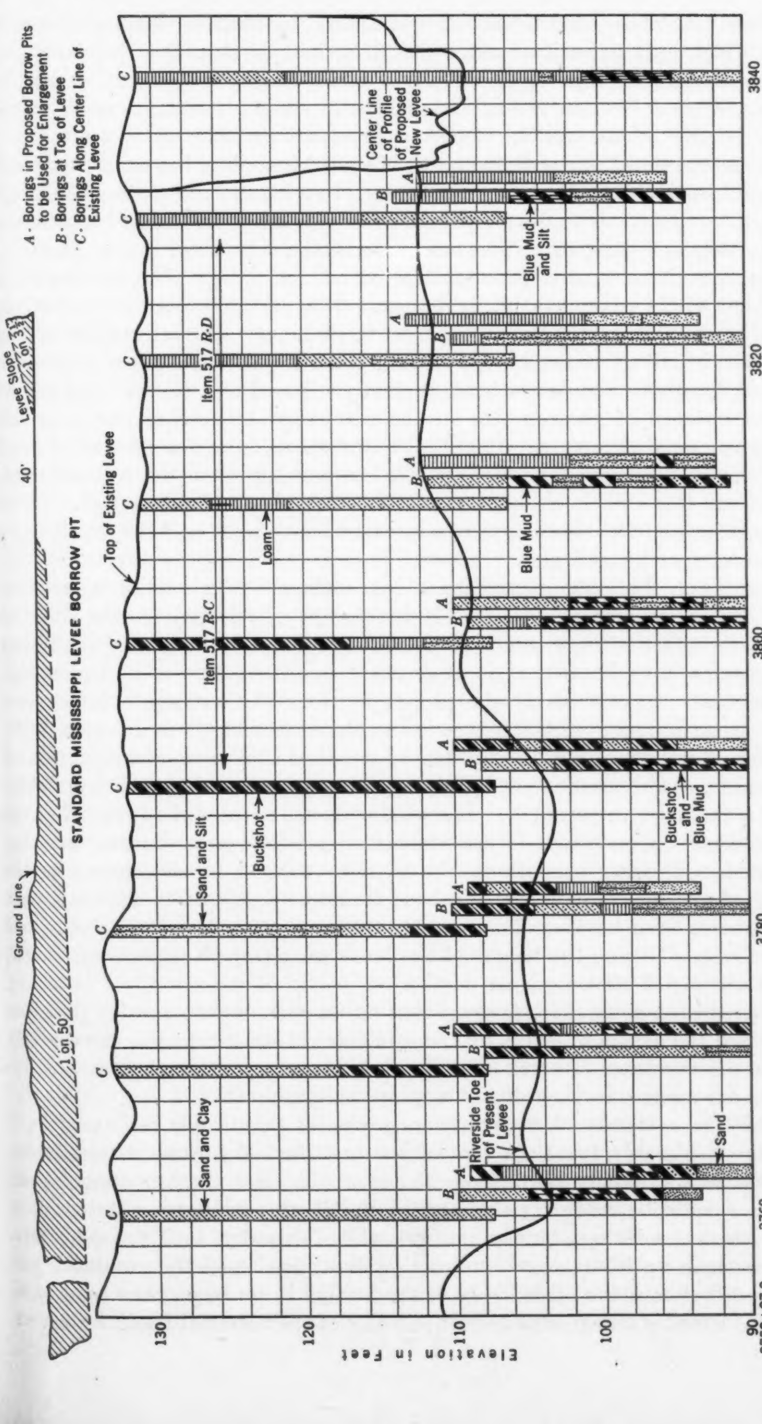

FIG. 65.—BORINGS BELOW GRAND LAKE

therefore, must be varied to the extent required by the river stage and water-table, which may necessitate much shallower borrow than that permitted by the specifications. Fig. 65 also shows a series of pit borings and groups of borings taken at the center line and the toe of a completed levee to illustrate the variability in composition of borrow, foundation, and embankment.

In general terms, levee design on the Lower Mississippi has concerned itself with compaction only from the standpoint of shrinkage allowance to insure a structure permanently to the grade and cross-section desired, and with moisture content only to the end that slides or sloughs shall not develop. As to character of materials, it recognizes three groups—sand, loam, and clay, as described in Table 1 of the paper by Mr. Buchanan. Most of the levees built are of the "loam" cross-section, "loam" being used to designate all mixtures containing less than 75% of either sand or clay. The loam cross-section has a base width equal to ten times the design head against the structure. It seeks ideally to prevent outcrop of seepage line on the rear slope by having the slope flat enough to contain the seepage line that will develop during the period of flood use. Of course, where the foundation is less pervious than the embankment, the seepage line will inevitably outcrop if the flood lasts long enough. Upon a stream such as the Mississippi it is essential that slopes be flat enough to be stable under such conditions.

In essence, the levee represents an embankment which relies upon great width rather than great density. It is feasible, physically, for the duty to which a levee is subject, to make the substitution, and in the case of the Lower Mississippi a consideration of local physical conditions and costs has caused that to seem unquestionably the proper course. The average contract cost of the approximately 567 000 000 cu yd so placed from 1928 to June 30, 1936, has been about 18.75 cts. It cannot be doubted that proper compaction, if practicable, would permit some reduction of cross-section where the foundation was of a character to permit it. It should be borne in mind, however, that the structure rests upon a foundation which is alluvial to great depths, and that the passage of water beneath the levee may cause as much concern as the passage of seepage through it. The base should be comfortably wide as a rule.

A levee is built between floods and is always more or less of an emergency undertaking. During the height of the levee program, placement was at the rate of about 612 000 cu yd per day.

From the foregoing it is apparent that unless current construction practices are revised the design of levees by the principles of soil mechanics can scarcely achieve the exactness of earth-dam design. However, although practical difficulties are numerous, they do not appear insurmountable.

With the methods of construction normally permitted, for example, a density considerably less than that of the undisturbed borrow material may characterize the freshly built levee but it would seem that in designing an allowance could be made for this condition. In this connection it is well to remember that soils high in clay content, when fairly dry, tend toward lumpiness, particularly if dragline construction is utilized and the resultant percentage of voids in the fill is likely (temporarily) to be large, thus tending to offset the effect of the greater impermeability of the material itself. Curve 1,

Fig. 64, illustrates such a case. This is a practical aspect of the matter which might prove troublesome if the width of cross-section were reduced too greatly on the basis of the impermeability displayed by borrow-pit samples. Obviously, the problem of design would be simplified if circumstances permitted compaction to be assumed or required. Where they do not so permit, caution is indicated in cases where the condition just described is likely to be encountered. Such levees improve rapidly with age but they must be ready and able to hold out floods the first year of their existence.

Study of borings in the borrow area will indicate the worst possible average composition that the levee can have. In deciding upon this feature and upon the questions raised by the preceding paragraph it would be well to consult some one familiar with the practical aspects of construction. It would also be well to consider, from knowledge of the local geology, the likelihood of encountering large enough deposits of any one unfavorable material to justify making that single type the sole basis of design.

The assumption of an impervious base should always insure an error on the side of safety in so far as seepage through the cross-section is concerned provided, of course, that the governing pit material is properly chosen and its observed permeability properly adjusted to compensate for lack of compaction. The partial or even complete disregard of cohesive strength in cases in which high water-content of cohesive borrow is at all likely should fully compensate for possible use of wet material in construction.

By familiarizing himself with local conditions, and adjusting his various allowances to meet them, the designer should be able to produce structures that will be both safe and economical. He should particularly guard against the temptation to produce too economical a structure, however, and, if possible, he should work "hand in hand" with the man of levee experience. Although the writer does not subscribe to the view that experience is the only guide in levee design, he is far from undervaluing it.

The designer cannot afford to ignore certain practical considerations not directly connected with soil mechanics. For example, levees should be maintained in sod, unless they are paved; hence the side slopes should be flat enough to permit the use of mowing machines, and to avoid excessive damage from rain and waves. If the structure is more than a few feet high the slopes for practical purposes should probably not be steeper than 3 horizontal to 1 vertical. In many cases, this may produce a section in excess of that indicated to be necessary by theoretical analysis.

The method of design outlined by Mr. Buchanan assumes an impervious foundation and a flood of great enough duration to develop fully the ultimate flow net. This is probably prudent on a stream such as the Lower Mississippi whose flood hydrograph may cover several months; but for lesser streams where the flood may gather and disappear in the course of a couple of weeks, or even days, it may result in over-design. The seepage behavior of the earth dam described by Mr. Buchanan indicates that for similar soils, similarly placed, the question of seepage can probably be omitted as a design criterion if the floods to be expected are brief, but other cases arise in which it seems unsafe to assume that the matter of seepage can be dismissed, yet in which the assump-

tion of full development of seepage seems too pessimistic. It is felt that further research regarding the rate at which the flow net develops is needed to "round out" the equipment of the levee designer.

The main-line levees on the lower river were matured as to general design before the science of soil mechanics was sufficiently advanced to lend much aid, and, as emphasized by Mr. Buchanan, analysis of the adopted design does not indicate any theoretical need for its revision. No doubt, soil mechanics will play an increasingly important part in the design of levees now in prospect as adjuncts to the basic system.

It has also found a useful field in connection with the prevention or correction of foundation failures in the main levee system. Contrary to popular opinion, most of the levee line experiences no serious foundation trouble. There is a consolidation of base in practically all instances, but this is not harmful unless it becomes excessive.

Serious trouble often threatens, however, where the levee crosses (as it unavoidably must at times) an old lake or channel that has filled with sediment. In such cases special treatment by way of flattened slopes or false berms is necessary and, in the opinion of the writer, the science of soil mechanics offers by far the most rational if, indeed, not the only rational, approach to the important matter of their correct design.

DONALD M. BURMISTER,[65] ASSOC. M. AM. SOC. C. E. (by letter).[65a]—The Symposium reflects the recent and important advances which have been made in the practical applications of soil mechanics to the study and design of earth structures. This new technique should include not only the analysis of earth structures for stability but it should carry the entire study of such problems from the preliminary soil investigations to the results of experiences accumulated during the construction of the work. The preliminary soil investigations on the different borrow-pit soils, from which suitable materials can be excavated economically, have an important bearing not only on the construction but also upon the decision as to the type of dam to construct, the dimensions of the sections, and the selection of stable slopes.

The grain size analysis, to which Mr. Dore refers, has constituted an important soil test on every earth dam project, but its value has been somewhat limited because the information obtained was, primarily, of a qualitative nature. The writer has found that such curves can also be adapted to yield valuable information of a quantitative nature, for design and control purposes.

Grading Curves.—The size characteristics of soil may be defined in terms of three independent factors, which correlate many physical facts and bring them into a more unified and consistent pattern: (*a*) The degree of fineness (the position of the grading curve on the scale of fineness); (*b*) the type of grading curve (normal, skewed, or irregular distribution of sizes); (*c*), the range of particle sizes. In his studies, the writer has expressed the degree of fineness in terms of a mean grain size, which is defined by the length of the equivalent rectangular area and which is easily obtained by planimeter or by the summa-

[65] Asst. Prof. of Civ. Eng., Columbia Univ., New York, N. Y.

[65a] Received by the Secretary January 12, 1938.

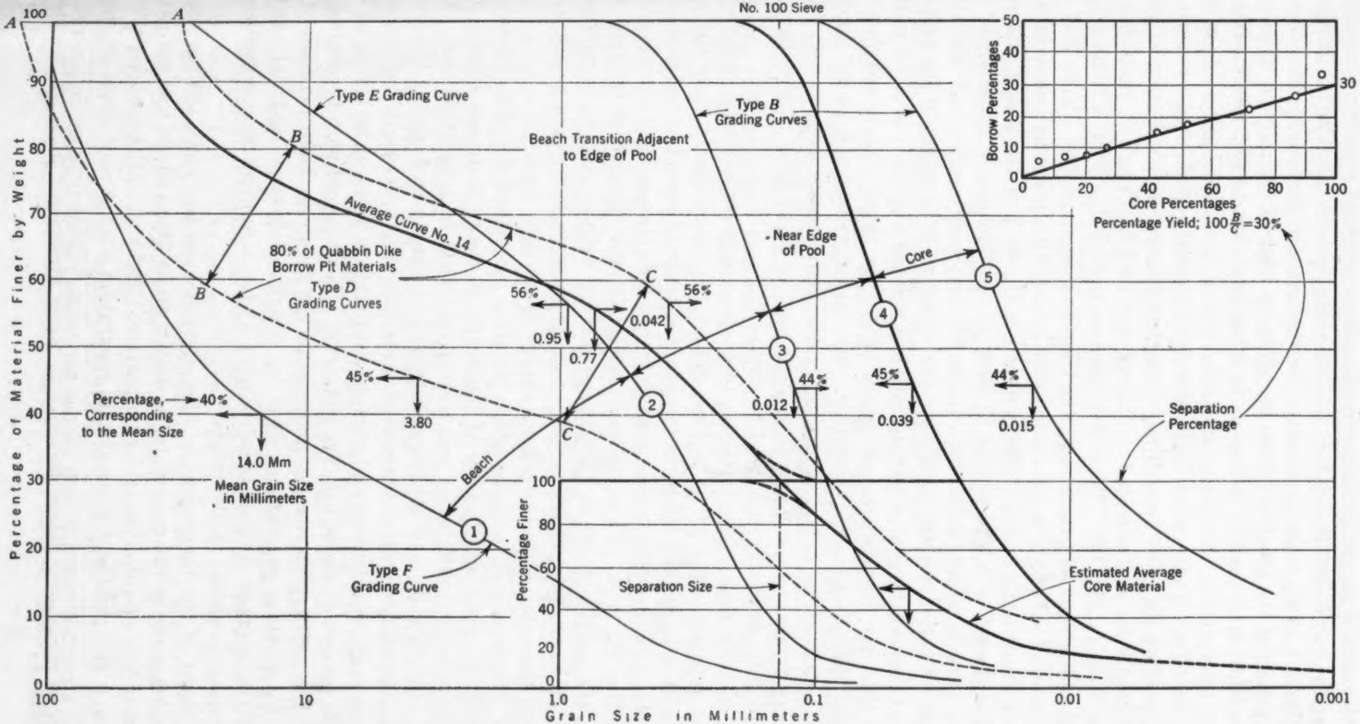

FIG. 66.—MATERIALS OF BORROW-PITS, BEACHES, AND CORE OF THE QUABBIN DIKE, SHOWING: (*a*) MEAN GRAIN SIZE; (*b*) CORRESPONDING PERCENTAGE; (*c*) TYPE OF GRADING CURVE; AND (*d*) ESTIMATION OF YIELD OF CORE MATERIAL

tion of elementary areas. This mean is actually a weighted geometric mean and is the logical one to use in relation to a semi-logarithmic plotting of the grain size analysis.[66] The mean grain sizes for the average borrow-pit, beach, and core materials of the Quabbin Dike are given in Fig. 66.

A description of grading curves as to type, as shown in Fig. 66, furnishes a basis for interpreting, correctly, the influence of the distribution of particle sizes on density. Type *A* is a narrow-range material represented by a nearly vertical grading curve. Type *B* has a statistical normal distribution of grain sizes, or nearly so, as illustrated in Curves 3, 4, and 5 of the core materials of Quabbin Dike in Fig. 66. Type *E* is predominantly fine with a little coarse material and is skewed toward the fine fractions, as shown by Curve 2 of the beach material. Type *F* is predominantly coarse with some fine material and is skewed toward the coarse sizes as shown in Curve 1 of the beach material. Type *D* material, which is typical of 80% of the borrow-pit materials of the Quabbin Dike is a mixture of two fractions, one very coarse, and the other quite fine. Such materials are always characterized by a hump in the grading curve and they are typical of certain soils in glaciated regions. It is interesting to note that the percentage corresponding to the mean size is not now an arbitrarily defined constant quantity, but varies with the type of grading curve in a characteristic fashion and in reality represents a skew factor, 45 to 55% being typical of the Type *B* grading curves, 55 to 65% of the Type *E*, and 35 to 45% of the Type *F* grading curves.

Soils may be alike in the foregoing important respects but may vary in the range of particle sizes. The writer, therefore, recommends that the size characteristics be defined finally in terms of the range of sizes, the designation of range to take in the entire curve being conveniently expressed as the ratio of the 95% to the 5% size. Thus, there is an important distinction between the shape of the grading curve and the range of sizes, which must not be confused. There is an interesting, characteristic, and progressive decrease in the range of sizes from the outer slope toward the center of the core pool.

Core Materials.—The sluicing-bin method described by Mr. Dore for estimating the yield and quality of the beach and core materials from a particular borrow-pit material or mixture of materials is a valuable one. The yield of core material may also be estimated by a simple analysis[66] of the aforementioned grading curves of borrow-pit materials, which, for eight dams, shows that there is quite a definite and complete separation point of materials by the hydraulic process at about the No. 100 sieve. The approximate percentage of yield of the average borrow-pit material of the Quabbin Dike is obtained in Fig. 66 by plotting percentages finer by weight of borrow material against those of the average core material (Curve 4) for a number of corresponding particle sizes, as shown in the insert.

The slope of the linear part of the curve defines the yield (30 per cent). The grading curve of the core material may then be fitted to the fine portion of the average borrow-pit material by proportional dividers, 30% becoming 100%, as shown. The yield and approximate separation size for nine dams are listed

[66] For a full description of such grading curves, see "A Study of the Physical Characteristics of Soils—with Special Reference to Earth Structures," by D. M. Burmister. (Not published.)

in Table 9. Thus, a study of curves of borrow-pit materials, for example, using the No. 100 sieve as the separation size, would indicate which materials to select or what mixtures to use in order to give the theoretical core width at any height, in the dam, as indicated in Fig. 18, as well as the approximate grain-size distribution of the average core material.

TABLE 9.—APPROXIMATE PERCENTAGE YIELD OF CORE MATERIAL

Item No.*	Name of dam (1)	Yield percentages (2)	Separation grain size, in millimeters (3)	Item No.*	Name of dam (1)	Yield percentages (2)	Separation grain size, in millimeters (3)
1	Henshaw†	41	0.16	10	Saluda‡	58	0.10
2	Swinging Bridge†	36	0.17	12	Alexander†	49	0.014
3	Dwinnell†	24	0.15	13	Cobble Mountain†	40	0.17
4	Davis Bridge‡	45	0.13	14	Quabbin Dike	30	0.14
9	Tieton‡	12	0.12	..	No. 100 sieve	..	0.148

* See Table 4. †Hydraulic-fill dams. ‡Semi-hydraulic fill dams.

A study of the data on core materials of existing dams in Fig. 16 reveals a number of significant facts: (1) The core materials of practically all existing dams are typically *B* Type: (2) all the core materials have about the same range of particle sizes, and the range is relatively narrow; and (3) the major distinguishing characteristics then are the degree of fineness as defined by the mean grain size and the clay content, which furnish a clue to the quality of the core material that can be washed out of any given borrow-pit material. It is important to note the characteristic gradation of the core material in Curves 3, 4, and 5 for the Quabbin Dike. However, physical factors other than grain-size distribution and fineness of the material passing the No. 100 sieve, may be of much greater importance. The liquid limit and plasticity index of the fine fraction furnish a more direct and useful index of the character and quality of the core material than can be obtained by the hydrometer test,[66] the practical limits being: Liquid limit, 15 to 40; and, plasticity index, 0 to 20. Values greater than these would indicate undesirable qualities in the core material and would indicate that difficulty might be experienced during, or subsequent to, construction; or it would dictate that fines should be wasted, where practicable.

Beach Materials.—The quantity of beach material that can be obtained from a given borrow-pit material can be estimated from the yield on the No. 100 sieve. The quality of the beach material, which is the most important consideration for stability of the entire structure, is determined, primarily, by the "grading density relations" of the materials deposited in the different sections of the dam. Arthur Casagrande,[67] Assoc. M. Am. Soc. C. E., has shown that the density of a material has an important influence on the angle of friction and on the stability of materials placed in embankments. The density of a given soil will lie between the limits of some loose condition and some very dense condition, whether placed artificially in an embankment by some mechanical

[67] "Characteristics of Cohesionless Soils Affecting the Stability of Slopes and Earth Fills," by Arthur Casagrande, *Journal*, Boston Soc. of Civ. Engrs., January, 1936.

equipment, or by the hydraulic process. Investigations of the grading-density relations[66] of materials show that the distribution of particle sizes exerts a most important influence, as noted in Fig. 67.

The mean grain size of each type of material, as the range of particle sizes varies, is indicated on the grading curve. The loose and dense conditions for each material are then plotted directly below on the vertical line through this mean grain size. The conclusions on the grading-density relations are summarized as follows:

1.—Degree of Fineness.—For the same type of grading curve, range of particle sizes, and particle shape, the density decreases with fineness approximately according to the trend of the zone lines.

2.—Type of Grading Curve.—Other factors being equal, Type *F* materials are invariably the most dense, and approach the ideal grading for maximum density. Types *B* and *E* are of intermediate density; and Type *A* materials are always the least dense.

3.—Range of Particle Sizes.—The most dense condition, other factors being equal, is obtained by the material having the greatest range of particle sizes. The narrow-range Type *A* materials are always the least dense.

4.—The Spread Between the Loose and Dense States.—There is a characteristic spread between the loose and dense states for each material.

5.—Particle Shape.—The effect of angularity of particles is always to decrease density in both the loose and dense conditions.

Fig. 67 is a graphical expression of the fact that each material can exist in a certain loose condition in its natural state or in embankments and can be compacted only to a certain dense state by the particular method of placing, with a large or small spread between them. The shearing characteristics of Type *F* materials, for example, are given in Fig. 67 to show the influence of these factors. The coarse, narrow-range Type *A* materials are practically always at about the critical density[67] with a relatively high angle of friction, of about 45°, but as the range of particle sizes increases, density has an important influence, the limiting values for the angle of friction being about 37° for the critical density and about 42° for the dense state. These values appear to be independent of the degree of fineness and of the range of particle sizes, where the range increases. The relation of the actual density in place, of a given type of material to its critical density will indicate which angle of friction will control the stability of the embankment. Professor Casagrande[67] has shown that the critical density-shearing relations are very important factors in stability, particularly for certain fine-grained, saturated, cohesionless soils.

A study of the beach materials of Quabbin Dike in Fig. 66 reveals a number of interesting and significant facts: (1) The coarsest fractions, which are deposited first, always contain some intermixed finer material; (2) the materials become progressively finer; and (3) the range of particle sizes becomes narrower toward the edge of the core pool.

The question may be asked, whether the gradations of beach materials in Fig. 66 (Curves 1, 2, and 3) are typical of the hydraulic process or whether they are typical of what the hydraulic process will separate out of the Type *D*

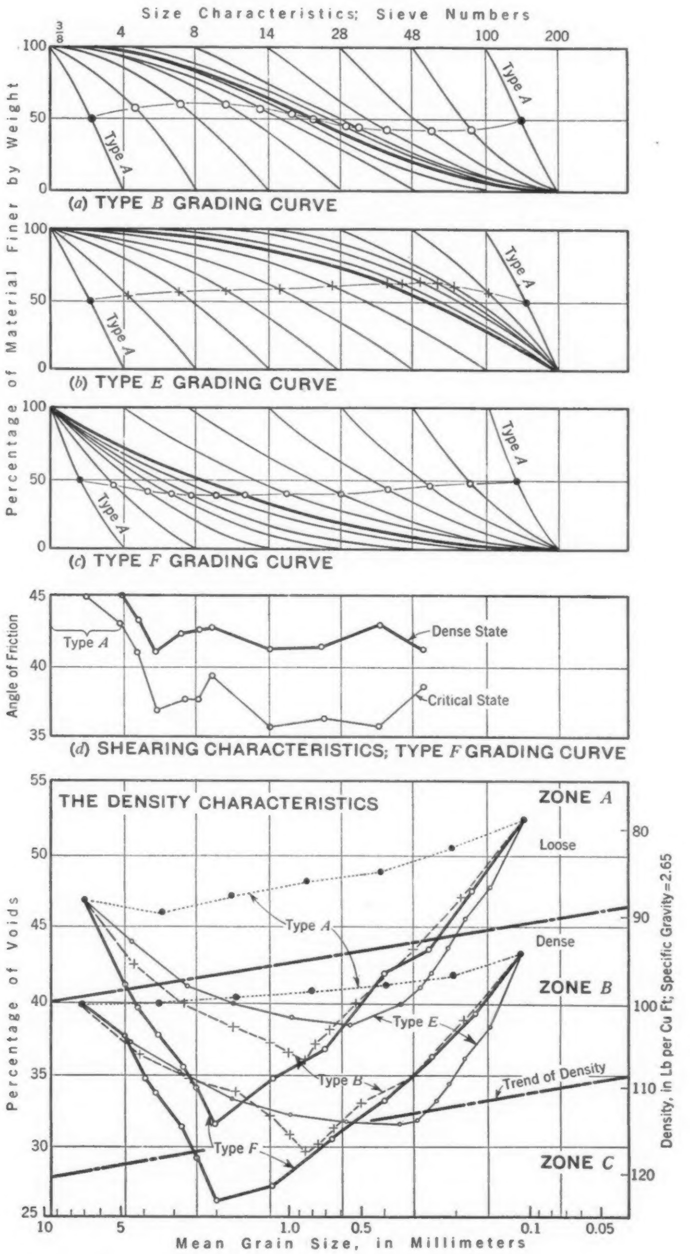

FIG. 67.—GRADING-DENSITY RELATIONS OF DRY GRANULAR MATERIALS, SHOWING EFFECT OF TYPE OF GRADING CURVE AND RANGE OF GRAIN SIZES

borrow-pit materials of Quabbin Dike. Keeping the foregoing three factors in mind and holding the No. 100 sieve point fixed as the lower limit of the range of particle sizes of the beach material, an upper limit is moved progressively along the 80% band of borrow-pit materials from the coarsest sizes in some proportion to the distance from the discharge point on the beach. From Point *A* to Point *B* the outer beach material must be typically of the coarse, wide range (*F* Type) and may vary from 20% to 50% of the bulk of the Type *D* borrow-pit material. It can be readily seen that this part of the curve is similar to Curve 1 and is of the *F* Type. The density for such material may be estimated from Fig. 67, by the trend of the zone lines, to be greater than 130 lb per cu ft as the tests by Mr. Dore show. This indicates that the hydraulic process places a Type *F* material in a very dense state. In the Quabbin Dike these materials form a substantial part of the entire beach.

When the coarse sizes are practically all deposited, the beach material must change in character from Point *B* to Point *C* to typically the finer *E* Type of material, having a density of about 115 to 120 lb per cu ft, and representing about 15% to 25% of the whole. Then, from Point *C* toward the edge of the pool the character of the material must change again and become typically the fine, narrow-range, *B* Type with an estimated density of 110 lb per cu ft, or less. It is evident that the character of the beach material is determined to a large extent by the type of grading curve of the borrow-pit material.

For stability considerations an angle of friction of 45° for the very coarse, Type *F* outer beach material, is to be expected, but as the materials grade into the finer *E* and *B* Types, the density not only decreases, but these finer materials may or may not be placed in a dense state by the hydraulic process and may even be less dense than the critical. The angle of friction may then be considerably less than 45°, and this would become an important consideration in the stability of the structure. However, in the Quabbin Dike an angle of friction of less than 30° was actually required in the beach material as a whole for stability, because the beach section was large.

It is believed that the information from soil investigations, together with the experiences accumulated in the construction of earth structures, when summarized, correlated, and made generally available, would be of permanent value as a guide for future earth dam construction. This means, however, the careful and complete description and identification of the materials used, and the determination of their density, shearing resistance in place, and behavior, so that the experiences gained in one locality may be of practical value in solving the problems in another.

DONALD W. TAYLOR,[68] ASSOC. M. AM. SOC. C. E. (by letter).[68a]—The civil engineer who has a general interest in soil mechanics will find much of value in the paper by Mr. Buchanan, which in addition touches upon a number of details of importance to those whose main interest is in this subject. Outstanding among such items is the effect of seepage forces upon stability of embankments. It may be noted that the effect of seepage may be handled by either of two methods which are closely related. It can be demonstrated that the vector

[68] Research Assoc. in Soil Mechanics, Mass. Inst. Tech., Cambridge, Mass.

[68a] Received by the Secretary January 26, 1938.

sum of the total weight (both soil and water) of any mass of saturated soil and the resultant of all water pressures along its boundary is equal to the vector sum of the submerged weight of the mass and the resultant of all forces due to seepage within it. The same relation may be re-stated as follows: The resultant boundary water pressure equals the vector sum of the resultant seepage force and full hydrostatic uplift. Mr. Buchanan has suggested the summing of seepage forces throughout the mass whereas other investigators have preferred the summing of boundary pressures. A paper[69] by the writer presented in 1937, suggests an approximate method for simple embankments which involves no complicated summations.

One statement which may be misleading appears in this paper (see following Fig. 3), concerning the force, F_f. The statement as given is true only if the figure, *e f d b*, is square. Since in general the figure is not square, it should be stated that "F_f is the force tending to cause displacement in the direction of flow, equal to h_f times the unit weight of water, times the average area surface, $\frac{1}{2}(f\,b + e\,d)$, of unit thickness."

[69] "Stability of Earth Slopes," by Donald W. Taylor, *Journal*, Boston Soc. of Civ. Engrs., July, 1937.

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DISCUSSIONS

THE DESIGN OF ROCK-FILL DAMS

Discussion

BY MESSRS. CHARLES H. PAUL, AND A. FLORIS

CHARLES H. PAUL,[7] M. AM. SOC. C. E. (by letter).[7a]—As would be expected, the author has presented, in convenient and concise form, a discussion of the more important features of design and construction of rock-fill dams, which should be of value to all engineers having direct or indirect interest in such structures. The writer is in general agreement with the author's conclusions, and with the precautionary remarks that appear throughout the text.

Under "Definition," an earth-fill backed by a rock-fill is eliminated from the rock-fill classification, which seems to be proper. The writer infers that a rock-fill faced with earth or gravel is also eliminated, and that raises this question: When a rock-fill section is stable in itself, and when an earth or gravel facing is used simply to secure water-tightness, should not such design be included under the rock-fill definition? The Minidoka Dam on Snake River, in Idaho, is an example, in which it was economical to face the rock-fill with gravel and earth, allowing this facing to wash into the voids of the rock as the placing of the facing automatically raised the water in the reservoir. That is a design that may not often be used, perhaps, but it seems to the writer that it should be classified as a rock-fill dam.

The paper states that the English Dam, built in 1856, was destroyed in 1883; it would be of interest to know how it was destroyed—whether deliberately for some reason, or by natural causes and, if so, what.

The writer would be inclined to give rock-fill dams more credit for stability than the author appears to do. Assuming good design and construction, it seems as if no question as to stability should arise, except perhaps in connection with overflow, and even in such cases it has been the writer's experience that rock-fill dams will stand considerable overflow without damage, or even danger, to the structure. It is fair to assume that the capacity and design of the spillway is an integral part of the design of a dam. Even in case of concrete

NOTE.—The paper by J. D. Galloway, M. Am. Soc. C. E., was published in October, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1937, by Messrs. Cecil E. Pearce, and H. B. Muckleston; and January, 1938, by Harold K. Fox, M. Am. Soc. C. E.

[7] Cons. Engr., Dayton, Ohio.

[7a] Received by Secretary January 17, 1938.

dams, it is often of vital importance to have sufficient spillway capacity to prevent overflow of the main structure, not necessarily because of danger of failure, but for other reasons. Assuming a spillway with sufficient capacity, which should be provided in any case, it is the writer's opinion that where conditions are favorable, a rock-fill dam takes its place, without handicap, among the various types of approved dam designs.

The writer would not absolutely prohibit the use of spillway gates. Whether or not they should be permitted would depend largely upon the design of the gates themselves, including their control mechanism; whether or not a reliable operating crew is to be maintained at the dam at all times; whether access to the spillway gate control is such that it would not be cut off in case of emergency; and the value or importance of gate control as a feature of the project. In many cases, spillway gates are being operated and maintained with perfect safety, and are fully justified.

Correction for *Transactions:* Table 1, Item No. 21, Column (8), should read: "1.2 : 1, 1.1 : 1; and, 1 : 1"; and page 1463, Lines 23 and 28, change "concave" to "convex."

A. FLORIS,[8] ESQ. (by letter).[8a]—In this clear, concise, and authoritative paper the statement is made that rock-fill dams resist water pressure by their own weight only. It is also true that in most rock-fill dams, the weight of water contributes to their stability. It is well known that the most important economic advantage in the design of multiple-arch dams is gained by sloping their arch barrels. The weight of water above these barrels compensates for the deficiency in the weight of multiple-arch dams against the water thrust.

With the exception of the older dams (Lower Otay[9] and East Canyon[10] constructed with a steel core-wall, and the Oued-Kébir Dam[11] with a hollow, multiple-arch core diaphragm), the dam section is made water-proof by means of wooden, masonry, concrete, or steel facing. Hence, assuming a water-tight facing and a cut-off wall at the up-stream face, the weight of water above the slope of the dam will add materially to the resistance of the structure against sliding.

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[8] Dipl.-Ing., Los Angeles, Calif.

[8a] Received by the Secretary January 24, 1938.

[9] "A Rock-Fill Dam with a Steel Heart-Wall at Otay, Cal.," by W. S. Russel, *Engineering News*, March 10, 1898, p. 157; also, "Reservoirs for Irrigation, Water Power, and Domestic Water Supply," by the late James Dix Schuyler, M. Am. Soc. C. E., New York, 1905, p. 19; and "Otay Rock Fill Dam Failure," by Charles Whiting Baker, *Engineering News*, February 3, 1916, p. 236.

[10] "A Rock-Fill Dam with Steel Core Across East Canyon Creek, Utah," by W. P. Hardesty, *Engineering News*, January 2, 1902, p. 14.

[11] "Core Wall in Rock-Fill Dam Tilts When Reservoir Fills," by the late F. A. Noetzli, M. Am. Soc. C. E., *Engineering News-Record*, November 3, 1932, p. 529; also, *La Révue Industrielle*, September, 1931; and "Algerian Dams of Placed Rockfill," by I. Gutmann, *Engineering News-Record*, December 2, 1937, p. 889.

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DISCUSSIONS

ECONOMICS OF THE OHIO RIVER IMPROVEMENT

Discussion

BY MESSRS. FRED LAVIS, O. SLACK BARRETT, AND EDMUND L. DALEY AND FORREST E. BYRNS

FRED LAVIS,[25] M. AM. SOC. C. E. (by letter).[25a]—The members of the Society are to be congratulated on the paper by Colonel Hall and the very illuminating data he has presented in regard to transportation on the Ohio River and the costs thereof. They are an important addition to the available data on this subject.

There are two points to be noted, however: First, his assumption that Government expenditures in the case of the improvements on the Ohio River are in the same class with those for ocean terminals and lighthouse service; and, second, that comparisons of costs of transportation between line terminals are a true criterion of comparison between rail and water-borne transportation.

The comparison with the provision of land beacons for the guidance of airplanes, while perhaps partly apropos, is not entirely so, because these beacons have to do with the carrying of the mails and the need of national defense to keep abreast of other nations in the development of aviation. They are also a necessary service for the actual military needs of to-day.

Improvements of harbors on the coast at the expense of the nation are justified because they affect the entire life of the nation and its people, unless they believe in, and can actually practice, an isolationist policy—which they neither do nor can. The coast line and harbors of the United States are the concern of the nation as a whole, not merely of those who live next to them, and, therefore, must be a charge against the Federal Government.

To take only two specific examples: The coffee on practically every breakfast table in the country, every morning, whether it be New York, Ohio, Virginia, Colorado, or anywhere else, must come in through an ocean terminal

NOTE.—The paper by C. L. Hall, M. Am. Soc. C. E., was published in October, 1937, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: December, 1937, by Eugene L. Grant, Assoc. M. Am. Soc. C. E.

[25] Cons. Engr., New York, N. Y.

[25a] Received by the Secretary December 28, 1937.

port. Automobiles made in Detroit, Mich., for export must be shipped out through an ocean terminal, etc. Ocean terminals really serve a national use.

It is quite understandable also that the lower reaches of some rivers can be included as part of the coast line, or as parts of, or extensions to, maritime harbors. The exact line of demarcation between coast line and inland waterways may be difficult to determine. Notwithstanding the long established legal authority of the Federal Government over all "navigable" waterways, it is difficult, however, to consider such improvements as those of the Ohio River other than local improvements or as beneficial to other than local interests. Colonel Hall's paper tends to confirm this.

Of course, the writer is not unmindful of the possible far-reaching effects of flood control, but that hardly affects the present discussion.

Comparisons of transportation costs between main line termini are not of much value; neither are ton-mile costs *per se* inasmuch as the river is almost sure to be longer than a parallel line of land transportation. The true criterion is the cost of transportation from producer to consumer. If coal is produced at a mine on the river, shot directly into barges, transported, and then consumed at another point on the river after direct transfer from barge to firebox, the river charge is an accurate measure of the cost of transportation, provided *pro rata* costs of construction and maintenance of river facilities are included.

Colonel Hall's paper indicates that these last named costs are not included, and he thinks their payment by the general public is offset by general public benefits. It is difficult to follow this reasoning, or to understand why a taxpayer in Portland, Me., Boston, Mass., New York, N. Y., Seattle, Wash., or San Francisco, Calif., should pay for these local benefits to people who happen to live alongside the improved channel of the Ohio River.

There is also to be considered the very strict regulation of rail rates and the limitation of return by Federal agencies on investments in railways which takes no account of this subsidized competition.

Leaving this subject, there still remains the question as to whether river costs (including the cost of the improvement) between termini are true measures of the transportation costs of the material carried on the river. Much of this material, it would seem, must have further charges for land transportation at either or both ends. A true comparison, therefore, can only be made by comparing all transportation charges, including loading and unloading en route, if any, between producer and consumer.

For any bulk freight such as that handled by river, this may be an important consideration. It might well be that the actual full cost of transportation of coal between off-river points might be less if it were handled from a railroad siding at a mine to a railroad siding in the plant where it is finally consumed.

There seems to be little doubt that the improvements on the Ohio River benefit somebody and provide low costs of transportation on some goods for some people. It does not, however, seem to be apparent from this paper, or from other available evidence, that this justifies taxation of others whose benefits are very small, or nothing at all. Moreover, the mere comparison of costs between main-line termini by one or more different methods of transportation is not a true measure of comparison of transportation costs.

O. Slack Barrett,[26] Esq. (by letter).[26a]—Complete and convincing as this paper seems to be, a careful study reveals the need of emphasis on several aspects of it.

History of the Ohio Valley Improvement.—Although the first dam was completed at Davis Island, in 1884 (as Colonel Hall states) there was no real canalization until years after 1895 when The Ohio Valley Improvement Association was organized. By the time the Panama Canal was opened to commercial navigation in August, 1914, only twelve dams had been completed on the Ohio River, and most of those between 1910 and 1914. The sum and substance of the matter is that these dams were located and built to facilitate navigation over some of the worst parts of the river, and served principally to expedite coal movements from Pennsylvania and West Virginia mines to markets, some of which were in the deep South. Even then, it will be noted, two factors seem to have been the motivating considerations, namely, cheaper transportation and wider markets.

In 1914, the war in Europe began at about the same time that navigation was opened in the Panama Canal. It is impossible to separate the effects of these two simultaneous events, but the four years of war demonstrated the necessity of adequate transportation facilities, and the Panama Canal, by shortening the commercial distance between the two coasts, increased the disadvantages suffered by industries in these inland valleys in competition with those situated at tide-water. The result was the popular demand for the completion of the improvement of the Ohio River, deepening and stabilizing channels in the Mississippi River, and the improvement of other waterways.

About ten years after the close of the World War, and fifteen years after the opening of the Panama Canal, the canalized Ohio River was open for continuous navigation. In the mean time, oil had largely replaced coal in the Southern markets, and what was left of the coal movement amounted to a number of important isolated trades. It is significant, however, that the prospect of dependable channels in the Ohio and Mississippi Rivers attracted the attention of alert industrial managers to such an extent that the 1929 traffic (not counting that of the tributaries) amounted to 1 513 000 000 ton-miles, with coal accounting for 50%, and the growing iron and steel traffic accounting for about 27 per cent.

The year, 1929, will long be remembered as the beginning of the great depression. By 1932, the traffic decreased to 1 392 000 000 ton-miles, but in 1933, with the resumption of industrial activity in the United States, the traffic began to mount until 1936 when it attained 2 653 000 000 ton-miles, representing an increase of 75.5% over the traffic in 1929, with iron and steel accounting for almost 34 per cent.

Benefits.—Colonel Hall has definitely shown, by detailed figures, the most authoritative estimate of the actual savings afforded by the present use of the improved Ohio River. This might be considered as the direct public benefit. There has been some speculation as to whether the savings are enjoyed by the consumer or by the producer, by the shipper or by the consignee. Such specula-

[26] Pres., Ohio Valley Improvement Assoc., Cincinnati, Ohio.

[26a] Received by the Secretary January 3, 1938.

tion is interesting from an academic viewpoint as an effort to determine what part of the public derives the benefit. In fact, both the producer or the shipper and the consumer or the consignee are the public, and any benefits derived by either or by both are derived by the public.

In addition, there are the indirect benefits merely suggested in several places in the paper. These benefits cannot be measured, charted, and definitely analyzed, but they probably constitute the greater part of the economic value of the improvement. It has been freely admitted that the great steel industry could not exist in the United States without cheap water transportation. It is certain that the Pittsburgh industrial area is dependent upon it. No man would hazard the suggestion that the public is not benefited by such industrial activity. What public-spirited citizen of large vision would advocate concentration of all the major industrial plants on the seaboard? The improved Ohio River is a large contributing factor in retaining and attracting industry, thus contributing to a more balanced development of the nation.

It is a definitely established fact that the rate structure applicable to commodities which form the bulk of the freight on the rivers has been substantially lowered. The best evidence on this point can be found in the rates approved by the Interstate Commerce Commission in the last decade. The freight bill paid by citizens has been lowered substantially.

The net result of the improvement of the Ohio and Mississippi Rivers is that the purchaser has been able to buy, and the seller to sell, in wider markets. More opportunities have been afforded both buyer and seller, all to the direct and indirect benefit of the citizens of the nation. The benefits are increasing with the growing use of the river. These were the ends on which the improvement was predicated.

EDMUND L. DALEY,[27] M. AM. SOC. C. E., and FORREST E. BYRNS,[28] ASSOC. M. AM. SOC. C. E. (by letter).[28a]—Too often the discussion of the economics of river improvements reaches one of two fallacious conclusions:

(*a*) Internal waterways are anachronistic. They have outlived their usefulness in the development of the country. Their further improvement by the United States is, in all cases, a waste of Federal funds.

(*b*) Improved waterways constitute the panacea of all transportation ills. They provide the minimum in transportation costs and, in addition to the savings procured to the public through shipments over the waterways themselves, they have a beneficial effect on the rates charged by all other carriers.

Colonel Hall avoids either extreme position. Dispassionately, he has analyzed the economic factors involved in modern transportation on one particular waterway.

In a study of the Ohio River, derivation of tangible benefits is made easier because of the abundance of accurate data on the volume and types of river traffic. As in all studies of this kind, the greatest difficulty lies in determining

[27] Col., Corps of Engrs., U. S. Army; Div. Engr., North Atlantic Div., New York, N. Y.

[28] Engr., U. S. Engr. Office, North Atlantic Div., New York, N. Y.

[28a] Received by the Secretary January 17, 1938.

the actual transportation costs to the shippers. The author follows a logical method of analyzing the complicated economic questions which must be considered.

All rivers are not worthy of improvement. Some rivers do have characteristics which constitute a *prima facie* case for economic river transportation. In the present day, river transportation is most useful in the movement of bulk commodities originating at points lying close to the river route and destined for points also close to the waterway. The Ohio River is especially well situated for manufacturers and shippers to take advantage of this condition.

Bulk commodities, such as coal and coke, iron and steel, oil and gasoline, stone, sand, and gravel, and cement, originate on or near the main river or its tributaries and move at low costs to points on or close to the river or connecting waterways. This is a natural and proper utilization of this national asset.

Colonel Hall analyzes the public benefits resulting from these low shipping costs. He limits the analysis to a discussion of transportation differentials and makes a careful comparison of the entire cost of the waterway shipping (including construction costs) with rail shipping. He finds that the total cost of water shipments on the Ohio River at the present time is less than the cost of rail shipments. In the past few years, traffic on the river has been increasing. The present trend indicates that both the tonnage and, more particularly, the ton-mileage will continue to increase. The conclusion reached is that from both an engineering and an economic standpoint, the canalization of the Ohio River has been successful and that this success tends to become greater with time.

The author is to be commended for the definite statement and discussion of the assumptions upon which much of his cost study is based. The most vulnerable point for criticism of this or any similar study is his Assumption (1). This is tacitly admitted by Colonel Hall, as well as the fact that any error in the assumption favors the waterway. An attempt in each individual case to determine the exact percentage of savings due to water shipments that is passed on to the public is clearly an impossibility. That the percentage is high and that, as Colonel Hall states, with the continued increase in the use of the waterway and the resulting keener competition, this percentage will continue to increase until eventually the original assumption will become accurate, is undoubtedly true.

Presumably, opponents of river improvement will grant that as far as the Ohio River is concerned, if these computed definite actual benefits are in error, at least the error is not great enough to change the results very much in the other direction. Then one comes to the consideration of other intangible public benefits that are difficult of assessment. Although these intangible benefits were not considered by Colonel Hall, they are very real.

Up to the time of the Civil War, traffic that was moved for any distance was largely water-borne. The development of the United States, and the extension of its settlement from the eastern seaboard, followed naturally along the navigable waters. Many of the early canals and early river improvements that were in themselves largely unprofitable were warranted by the development of the country which resulted from them.

Since the Civil War, the transportation picture has changed. As canal-boats were displaced by railroads and other more rapid means of transportation, commercial navigation on rivers and canals gradually assumed less and less importance.

Before this, due to the advantage of low-cost water transportation of bulk commodities, new basic industries had arisen. Large towns and cities had been built around these and related industries, with consequent economic benefit to the Federal and State Governments. Railroads have partaken of these benefits by reason of the high tariff freight which has been developed from new communities, such as Clairton, Aliquippa, and Ambridge, Pa., and Weirton and Follansbee, W. Va. These are very large benefits. New wealth was created. The monetary value of these benefits is positive but not definitely calculable.

Within the past few years manufacturers and shippers have begun to rediscover the advantages of shipping on the inland waterways. New equipment is being developed for the transportation of bulk commodities. Barges and self-propelled motor ships (especially tankers) are being designed particularly to fit the conditions existing on the more important inland waterways.

Time only will tell definitely whether the more recent improvements on the rivers of the United States, involving the expenditure of many millions of dollars, are in all cases economically justified. Colonel Hall's paper shows that the present commerce on the Ohio River and the resultant savings in transportation costs justify the large expenditures made for its improvement.

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DISCUSSIONS

SOLUTION OF TRANSMISSION PROBLEMS OF A WATER SYSTEM

Discussion

BY MESSRS. HAROLD E. BABBITT, AND C. MAXWELL STANLEY

HAROLD E. BABBITT,[12] M. AM. SOC. C. E. (by letter).[12a]—Problems involving the hydraulics of flow in conduits in series, in parallel, or in a network, are commonly met in the study of water distribution systems. Problems involving the flow in conduits in series or in parallel, or in combinations of conduits in series and in parallel, have been subject for some time to accurate and practical solution by mathematical analysis but, until recently, no method has been available for the solution of problems involving cross-overs or networks; unless Mr. Freeman's graphical method[13] may be so considered. This method involves so much labor for the solution of a practical problem that it has not been widely used. The electrical analogy, introduced in 1934 by Thomas R. Camp and the late Allen Hazen,[14] Members, Am. Soc. C. E. offers another method of solving such problems. It has not, however, been widely used, possibly because of the equipment required and its rather recent introduction. However, the author is not correct in his statement (see "Introduction") that "a mathematically accurate determination of the results to be expected in designing intricate grid systems or important reinforcements is impossible of attainment from a practical standpoint." Professor Cross' method for the analysis of flow in networks and conductors[15] makes it possible to solve such problems mathematically to any desired degree of precision, and results, using this method, are reported within 1% of accuracy by James J. Doland, M. Am. Soc. C. E. in his illustrative problem which the author has quoted.[4]

NOTE.—The paper by Ellwood H. Aldrich, M. Am. Soc. C. E., was published in October, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1938, by Messrs. Lynn Perry, Charles M. Mower, Jr., and Thomas R. Camp.

[12] Prof., San. Eng., Univ. of Illinois, Urbana, Ill.

[12a] Received by the Secretary December 15, 1937.

[13] *Journal*, New England Water Works Assoc., Vol. 7, p. 49, 1892.

[14] "Hydraulic Analysis of Water Distribution Systems by Means of an Electrical Analyzer," *Journal*, New England Water Works Assoc., Vol. 48, p. 383, 1934.

[15] *Bulletin 286*, Eng. Experiment Station, Univ. of Illinois, 1936.

[4] "Simplified Analysis of Flow in Water Distribution Systems," *Engineering News-Record*, Vol. 117, No. 14, p. 475, October 1, 1936.

Until the publication of the Cross method problems involving the hydraulics of network distribution systems were most commonly solved by guesswork or by approximate methods whose results were little more than guesswork. The author's method is a simplification and an extension of Freeman's graphical method, which is applicable to some special conditions of flow in networks. It could probably be extended to include all possible problems in such networks.

Problems that may arise in a study of the hydraulics of a network distribution system may be illustrated on the simple cross-over diagram shown in Figs. 13(*a*) and 13(*b*), in which the diameters and lengths of all pipes are known. The principles are applicable to any number of pipes and cross-overs in the network.

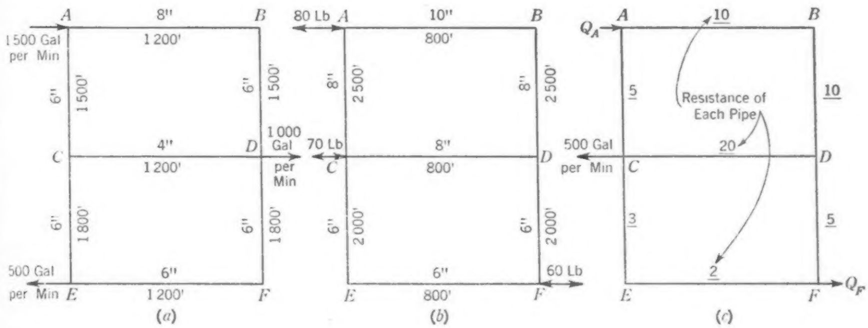

FIG. 13

Problems of Type I are illustrated in Fig. 13(*a*), in which all put-ins and take-outs are known, and it is desired to determine the distribution of flow in all the pipes and the pressures at various points in the pipe system. This can be solved by the Cross "Method of Balancing Heads."

Problems of Type II are illustrated in Fig. 13(*b*), in which the pressures are known at all points of put-in and take-out, and it is desired to determine the rate and direction of flow in all pipes and the rate of all put-ins and take-outs, and the pressures at other selected points in the distribution system. This can be solved by the Cross "Method of Balancing Flows."

Problems of Type III involve a combination of the conditions and unknowns in Types I and II.

The second type of problem is commonly met in water-works practice as, for example, where the pressures at a hydrant and at the pumping station, or at one or more distribution reservoirs, are known and it is desired to know the rate and direction of flow through the pipes in the distribution system. It is to be noted that where the Cross method is used for the solution of such a problem a change in one or more of the assumed pressures, put-ins, or take-outs, will involve the complete reworking of the solution, whereas if there is no change of the pipes in the distribution system, the problem resulting from changes in the conditions can be solved relatively easily by the method suggested by the author.

It is possible, however, that a number of problems, with successively slight changes in the conditions, might be solved by the Cross method, the results being plotted graphically with little or no more effort than would be involved in a series of solutions by the author's method. For example, let it be desired to determine the magnitude and direction of flow in each pipe in the simple cross-over system illustrated in Fig. 13(*c*). All the water enters at Point *A*; 500 gal per min is constantly taken out at Point *C*, and the take-out at the hydrant at Point *F* may vary between zero and 1 000 gal per min. The method suggested by Professor Doland has been used in this solution and, for simplicity, it has been assumed that $H = K\,Q^2$, in which H is the head loss in the conduit; Q is the rate of flow in the conduit; and K is a resistance factor dependent upon the length and diameter of the pipe. If the more commonly used Williams and Hazen formula were used here, the expression would be $H = K\,Q^{1.85}$. The computations involved in the solution are shown in Table 5 and Fig. 14, and the flows in each pipe, to the nearest 1% of accuracy, are entered in Table 6. These values can be plotted so that the rate of flow in any pipe, corresponding to any rate of flow from Point *F*, can be read from the curves. In Fig. 14, the underlined values denote relative resistances and the values in circles are the first assumptions of the percentage flow. The values beneath the circled numbers indicate corrections. The arrows in Table 6 indicate when the direction of flow in Fig. 13(*c*) is from left to right. When no arrow is shown the direction of flow is away from Point *A*.

Where added conditions are involved, such as storage reservoirs on the distribution system, or where the distribution system is on two or more levels, as on different floors of a multi-storied building, Professor Cross' method can be applied. There is no limit to the number of problems which may arise in the hydraulics of flow in a network distribution system. Any problem in this field can be solved by the Cross method and many problems can be solved by

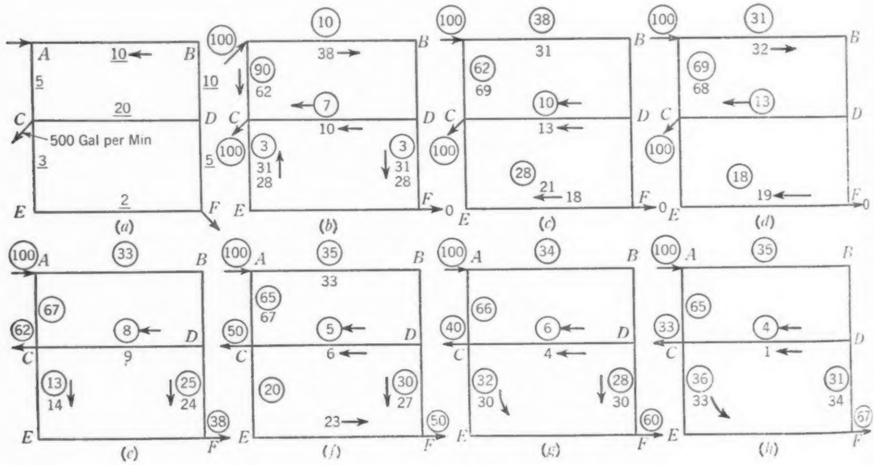

FIG. 14.—CONDITIONS OF FLOW FOR SOLUTION IN TABLE 5

TABLE 5.—COMPUTATIONS TO DETERMINE DISTRIBUTION OF FLOWS IN THE PIPES SHOWN IN FIG. 13(c), WITH TAKE-OUT AT POINT C = 500 GALLONS PER MINUTE, AND DIFFERENT TAKE-OUTS AT POINT F

Pipe* (1)	Resistance, K (2)	Flow, Q (percentages) (3)	$K\,Q^n$† (4)	$K\,Q^{n+1}$ (5)	$\Sigma\,(K\,Q^{n+1})$ (6)	ΔQ (7)	Flow, Q (percentages) (3)	$K\,Q^n$† (4)	$K\,Q^{n+1}$ (5)	$\Sigma\,(K\,Q^{n+1})$ (6)	ΔQ (7)
Q_F = 0; FIRST TRIAL (SEE FIG. 14(b))							Q_F = 305; ONLY TRIAL (SEE FIG. 14(e))				
ABD	20	10	200	2 000			23	660	21 800		
DF	5	3	15	45	2 045		25	125	3 125	24 925	
AC	5	90	450	40 500			67	335	22 400		
CEF	5	3	15	45	40 455		13	65	845	23 245	
Σ			680 × 2 = 1 360			$\frac{38\,410}{1\,360} = (28)$		1 185 × 2 = 2 370			$\frac{1\,680}{2\,370} = (1)$
CD	20	7	140	2 800			8	160	1 280		
DF	5	31	155	775	2 025		25	125	3 125	1 845	
CEF	5	31	155	4 800	4 800		13	65	845	845	
Σ			450 × 2 = 900			$\frac{2\,775}{900} = (3)$		350 × 2 = 700			$\frac{1\,000}{700} = (1)$
Q_F = 0; SECOND TRIAL (SEE FIG. 14(c))							Q_F = 500; ONLY TRIAL (SEE FIG. 14(f))				
ABD	20	38	760	28 900			35	700	24 500		
DF	5	28	140	3 920	32 800		30	150	4 500	29 000	
AC	5	62	310	19 200			65	325	21 100		
CEF	5	28	140	3 920	15 280		20	100	2 000	23 100	
Σ			1 350 × 2 = 2 700			$\frac{17\,540}{2\,700} = (7)$		1 275 × 2 = 2 550			$\frac{5\,900}{2\,550} = (2)$
CD	20	10	200	2 000			5	100	500		
DF	5	21	105	2 210	210		28	140	3 930	3 430	
CEF	5	21	105	2 210	2 210		22	110	2 420	2 420	
Σ			410 × 2 = 820			$\frac{2\,420}{820} = (3)$		350 × 2 = 700			$\frac{1\,010}{700} = (1)$
Q_F = 0; THIRD TRIAL (SEE FIG. 14(d))							Q_F = 750; ONLY TRIAL (SEE FIG. 14(g))‡				
ABD	20	31	620	19 200			34	680	23 100		
DF	5	18	90	1 620	22 400		28	140	3 930	27 030	
AC	5	69	345	23 800			66	330	21 800		
CEF	5	18	90	1 620	22 180		32	160	5 130	26 930	
Σ			1 145 × 2 = 2 290			$\frac{220}{2\,290} = (1)$		1 310 × 2 = 2 620			$\frac{100}{2\,620} = (1)$
CD	20	13	260	3 380			6	120	720		
DF	5	19	95	1 800	3 380		28	140	3 920	3 200	
CEF	5	19	95	1 800	3 600		32	160	5 130	5 130	
Σ			350 × 2 = 700			$\frac{220}{700} = (1)$		420 × 2 = 840			$\frac{1\,930}{840} = (2)$

* See Fig. 14. † n = 1. ‡ Computation for Q_F = 997 (Fig. 14(h)) has not been included.

the author's method. Where both methods are applicable to the solution of a problem the choice will depend on the experience and the judgment of the computer.

Some confusion may result from the use, by the author, of the term, "hydraulic grade," to represent the piezometric head or the internal pressure at a point in a conduit. The term, "grade," is usually synonymous with

TABLE 6.—RATES OF FLOW IN PIPES IN FIG. 13(*c*) FOR VARIOUS VALUES OF Q_F
(Values of Q Are Given in Gallons per Minute)

Pipe	Q_F = ZERO		Q_F = 305		Q_F = 500		Q_F = 750		Q_F = 997	
	Percentages	Q	Percentages	Q	Percentages	Q	Percentages	Q	Percentages	Q
ABD..	32	160	33	270	33	330	34	425	35	525
DF....	18	90	24	190	27	270	30	375	34	510
AC....	68	340	67	540	67	670	66	825	65	975
CEF...	19←	95←	14	110	23	230	32	400	34	510
CD....	13←	65←	9←	72←	6←	60←	4←	50←	1←	15←

"slope" rather than with "elevation" in most texts on hydraulics. The term, "pressure," or "head," would be more significant of the author's meaning.

C. MAXWELL STANLEY,[16] ASSOC. M. AM. SOC. C. E. (by letter).[16a]—Being useful in the solution of the complex problems involved in the design of systems for the transmission and distribution of water, the method suggested by Mr. Aldrich is both interesting and valuable. Methods of design for such systems have had too little attention from water-works engineers, who have usually considered the design of a distribution system as something too complex for accurate computation, and yet too simple and practical to warrant much study. As a result, the art of designing transmission and distribution systems has developed slowly and has been out-distanced by the developments in supply, treatment, and conditioning of water. The distribution system is easily overlooked as it has usually grown bit by bit and seldom does the water-works man realize the accumulated investment that lies beneath the streets—out of sight and considered only when necessary.

The author's statement that the transmission and distribution "elements of a water system commonly exceed 50% of the total value" is conservative. Table 7, pertaining to distribution in several municipal water systems in Iowa, indicates an average of more nearly two-thirds and in only one case does the ratio drop below 50%, in spite of the varying size and types of supply studied.

The graphical solution outlined by the author seems to be much better suited to problems of transmission or distribution which involve not more than a few parallel circuits, than to complicated grids with numerous circuits or cross-connections. The solution of grids by the methods of Case VI is very ingenious, but is laborious and time-consuming as compared to the method devised by Hardy Cross,[17] M. Am. Soc. C. E. Indeed the example contained in the paper as a "Solution of a General Case" seems in itself proof that the Cross method is more quickly and easily applied to networks. The solution of this problem, as presented by J. J. Doland,[4] M. Am. Soc. C. E., is more concise and much quicker than the solution presented by the author. The graphical solu-

[16] Cons. Engr., Young & Stanley, Inc., Engrs., Muscatine, Iowa.

[16a] Received by the Secretary January 21, 1938.

[17] "Analysis of Flow in Networks of Conduits or Conductors," by Hardy Cross, *Bulletin 286*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[4] "Simplified Analysis of Flow in Water Distribution Systems," by J. J. Doland, *Engineering News-Record*, October 1, 1936, p. 475.

tion becomes too involved with a complicated grid. Furthermore, a grid or network is generally analyzed for a maximum condition, usually that of normal consumption plus a fire flow located at some particular point which, for the particular section of the system under consideration, will give the worst condition of pressure. Thus, usually one condition only is studied and the work required to prepare the curves for each section of pipe for the graphical method is excessive. This fact further tends to limit the usefulness of the method to transmission problems.

TABLE 7.—PERCENTAGE OF TOTAL INVESTMENT IN DISTRIBUTION

System (1)	Population served, in thousands (2)	Supply (3)	Treatment* (4)	Percentage investment in distribution (5)	System (1)	Population served, in thousands (2)	Supply (3)	Treatment* (4)	Percentage investment in distribution (5)
A..	1.1	Deep wells	None	49.5	G.	17.0	Shallow wells	None	81.5
B..	1.5	Deep wells	None	78.3	H.	22.0	Deep wells	A, Ch	66.5
C..	4.1	Deep wells	None	84.5	I..	29.0	River	C, S, F, Ch, So	64.5
D..	6.6	Reservoirs	C, S, F, Ch	40.8	J..	42.0	Deep wells	Ch	64.5
E..	9.0	Impounding reservoirs	A, C, S, F, Ch	55.0	K.	48.0	Wells	A, S, Ch	66.0
F..	12.0	Shallow wells	Ch	58.2	L.	145.0	Infiltration galleries	Ch	69.0

* Key to symbols on treatment: A = aeration; C = coagulation; S = sedimentation; F = filtration; Ch = chlorination; and, So = softening.

The graphical method appears well adapted to problems with varying rates of flow. In such cases, the curves for different pipes and groups of pipes present a visual analysis of the happenings with varying flows. Direct mathematical computation, on the other hand, presents only the answer for a given condition and the effect of varying flow is not so easily visualized.

The author's method is exceptionally well adapted to problems involving storage (similar to Case IV) and those involving transmission mains, interconnecting sources of supply, pumping stations, and storage reservoirs or tanks. In such problems, the graphical picture of the characteristics of the system permits a simple showing of water elevations, friction losses, and quantities which is extremely helpful. It seems to the writer that it is for problems of this type that the graphical method is best suited and will find its widest application and use. The method also has a value in that it may be applied to small sections of a large system when it is desired to obtain a clearer "picture" of the relationship of flow, friction, and head with varying conditions. Often, a chart or graph as a supplement to computations affords a better understanding of the situation.

The foregoing comments apply to the general usefulness and application of the method. A few comments may also be made on some of the details.

In the list of "Basic Data" required there should be added under Group (*d*)—which deals with storage—the item of depth of storage. Such depth plays a large part in the value of a given storage volume. The effectiveness of

a given total volume in an elevated tank, as compared to the value of the same volume in a stand-pipe, is a good example of the need for considering depth of storage as an important factor.

Conversion factors for various values of C in the Williams-Hazen formulas for application to the friction losses obtained from Fig. 1 ($C = 100$), are, as follows:

C	Correction factor	C	Correction factor
70	1.94	110	0.84
75	1.70	115	0.77
80	1.51	120	0.71
85	1.35	125	0.66
90	1.21	130	0.62
95	1.10	135	0.58
100	1.00	140	0.54
105	0.91	145	0.50
		150	0.47

Conversion factors may be more readily obtained from such a list than from the extra curves, in Fig. 1, showing the relationship of $S^{0.54}$ and $S^{\frac{1}{0.54}}$.

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DISCUSSIONS

INCREASING THE TRAFFIC CAPACITY AND SAFETY OF THOROUGHFARES

A SYMPOSIUM

Discussion

BY MESSRS. JOHN A. OAKEY, L. W. MAHONE, FRED LAVIS, HUGH A. KELLY, AND SIDNEY J. WILLIAMS

JOHN A. OAKEY,[16] ASSOC. M. AM. SOC. C. E. (by letter).[16a]—Work such as that described in this Symposium does much toward establishing traffic engineering as a practicable science. However, there is one item in the section dealing with the "Economics of Alignment, Grade, and Width," as presented by Mr. Schmidt which, in the writer's estimation, requires further consideration. That item is the cost of operating a motor vehicle.

Ever since Agg and Carter[14] published the results of their investigation on the cost of operating a passenger car, these results have been accepted with little or no consideration of the fact that motoring costs have undergone radical changes since 1928. These findings, based upon an annual travel of 11 000 miles, are presented in Table 15, Columns (2), (3), and (4).

The results of studies[15] by Mr. Johannesson on the cost of operating a passenger car, are shown in Table 15, Column (5). These values are based upon an annual travel of 10 000 miles and show a total somewhat in excess of those of Agg and Carter. In other words, according to these authorities, the annual vehicle cost for the average motorist lies between $700 and $750. A momentary consideration of the fact that the average motorist has an annual income surely not exceeding $2 400 seems ample proof of the fact that motoring costs cannot be this high. If they were, there would not be 25 000 000 passenger cars in operation in the United States.

NOTE.—The Symposium on Increasing the Traffic Capacity and Safety of Thoroughfares was presented at the meeting of the City Planning Division, Pittsburgh, Pa., October 15, 1936, and published in November, 1937, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the Symposium.

[16] Instr., Civ. Eng. Dept., Columbia Univ., New York, N. Y.

[16a] Received by the Secretary November 24, 1937.

[14] *Bulletin 91*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa.

[15] "Highway Economics," by Sigvald Johannesson, M. Am. Soc. C. E., 1931.

The writer has no experimental data to support his contention that these costs are excessive other than his own experience and that of his acquaintances, and certain statistics published by the Automobile Manufacturers' Association.[5] It is interesting, nevertheless, to study each item in Table 15, Column (1), from a common-sense viewpoint and with full consideration of the habits of the average motorist of to-day.

TABLE 15.—COMPARISON OF OPERATING COSTS, IN CENTS PER MILE

Description	Agg and Carter			Johannesson	Writer
	High type of roads	Intermediate type of roads	Low type of roads		
(1)	(2)	(3)	(4)	(5)	(6)
Fuel	1.09	1.31	1.61	1.39	1.2
Lubricants	0.22	0.22	0.22	0.20	0.1
Tires and tubes	0.29	0.64	0.84	0.75	0.3
Maintenance	1.43	1.72	2.11	2.03	0.5
Depreciation	1.26	1.39	1.57	1.73	1.0
License	0.14	0.14	0.14	0.19	..*
Garage	0.44	0.44	0.44	0.44	0.5
Interest	0.36	0.36	0.36	0.53	0.1
Insurance	0.21	0.21	0.21	0.23	0.1
Taxes	....	....	....	0.01	0.2†
Total	5.44	6.43	7.50	7.50	4.0

* Included in taxes. † Includes license fees.

(*1*) *Depreciation.*—In 1936, the average passenger car had an f.o.b. price of $603. The average retail price, therefore, was approximately $750. This car travels about 9 000 miles per yr and has a life slightly greater than 8 yr. It is fair, therefore, to assume that the life is 75 000 miles. On this basis, the depreciation is 1.0 cts per mile (see Table 15, Column (6)).

(*2*) *Gasoline.*—The average cost per gallon of gasoline was 18.84 cts in 1936. Assuming that the average car makes 15 miles to the gallon, the fuel cost is 1.2 cts per mile. The values in Table 15, Columns (3) and (5), based upon the same mileage per gallon but upon a fuel cost of 20 cts per gal, are, therefore, quite reasonable. The annual gasoline consumption of the average passenger car is 600 gal, giving a total annual travel of about 9 000 miles.

(*3*) *Lubrication.*—The average motorist changes the oil in his car between the 1 000-mile and the 2 000-mile mark. Assume that he changes the oil at the 1 500-mile mark, that the change requires 6 qt of oil costing 25 cts per qt, and that he finds that he requires an additional quart between changes. Assume further that he has his car greased every 3 000 miles at a cost of 50 cts. These assumptions place the total cost of lubrication at $2.00 per 1 500 miles, or 0.13 ct per mile. However, since no consideration has been made of the fact that many thousands of motorists

[5] "Automobile Facts and Figures," Automobile Manufacturers' Assoc., 1936 Edition.

service their own cars with chain-store products at a considerable saving, this item may reasonably be reduced to 0.1 ct per mile.

(4) *Tires and Tubes.*—The modern light-weight average car can be equipped with tires and tubes at a cost not exceeding \$50. These tires will have a life between 15 000 and 20 000 miles. On this basis, the tire cost will be slightly less than 0.3 ct per mile. However, the slight additional cost of repairs will bring this item up to 0.3 ct per mile.

(5) *Maintenance.*—The writer has driven his present car almost 100 000 miles at a total maintenance cost of slightly less than \$200. The resulting maintenance cost of 0.2 ct per mile, while simply the result from one car, appears as reasonable as the other values shown in Table 15. The latter represent annual total maintenance costs of \$189 and \$203, respectively, and are obviously much too high. Acquaintances of the writer have reported maintenance costs from \$5 to \$50 per 10 000 miles. In any case, an arbitrarily assumed annual maintenance charge of \$45, or 0.5 ct per mile should be sufficient to properly maintain the modern average car.

(6) *Interest.*—For some unknown reason, whenever an engineer is confronted with a problem involving economics, he writes down as his first item of annual cost "6% on the investment." To the average motorist, his car is no more of an investment than are the clothes he wears. The argument is frequently raised that, if he had not purchased his car, the motorist could have put his money out at interest. The apparent logic of this argument is "exploded" when one considers that the motorist would get, at most, 3% from a savings bank and that 60% of all new cars and 65% of all used cars are purchased on the instalment basis. Apparently, a large percentage of the motoring public in this country lacks either the ability or the inclination to save. The dialogue composed of: "Now that you own a car, you'll be broke all the time," and "I'm broke all the time anyway so I might as well have the car," is indicative of the national habit of owning a car at all costs. For these reasons, a nominal charge of 1% on the investment, or 0.1 ct per mile, seems all that can be justified on this item.

(7) *Taxes.*—Passenger car and truck owners to the number of 26 200 000 paid \$396 000 000 for registration fees and personal property, city, and county taxes in 1935. The average cost, with no attempt to segregate the two types of vehicles, is \$15. This figure results in a cost of somewhat less than 0.2 ct per mile.

(8) *Garages.*—Storage charges vary from \$15 per month in thickly populated urban areas to nothing in rural areas. The commonly accepted charge of \$4.00 per month, resulting in a cost of 0.5 ct per mile, is reasonable.

(9) *Insurance.*—In 1935, the owners of the 26 200 000 passenger cars and trucks in the United States paid a total of \$446 000 000 for all forms of automobile insurance, \$242 000 000 of which were returned in losses paid. The net charge of \$204 000 000 results in a cost of \$8.00 per vehicle, or 0.1 ct per mile.

(*10*) *Miscellaneous.*—The writer is loath to venture an opinion as to whether such items as "time" and "driver's wages" can be given a definite value. The arguments from authorities on either side seem to be equally balanced. Abstract time, to the average motorist, has little value, except as a convenience. When weighing the respective values of time and mileage, he usually considers the cost of gasoline as his only mileage charge. Whether engineers have the right to make him pay for a saving he does not appreciate is a question which may be argued at great length.

From a consideration of the tangible items comprising the cost per mile, it would appear that this cost is about 4.0 cts per mile for the average passenger car. Items (6), (7), (8), and (9) and, to a certain extent, Item (1), are relatively unimportant since these items vary with time and are not affected by any saving in distance or betterment in type of surface. If one-third of the depreciation may be charged to time and two-thirds to usage, the saving resulting from a reduction of distance is 2.8 cts per mile. This is considerably lower than the commonly accepted values.

There is another aspect to the reduction of distance which is seldom considered. This is the effect of the relative length of the reduction. In railroad work, a sliding percentage is applied depending upon the length of the reduction. Unfortunately, it is impossible to break down motor-vehicle costs in a manner comparable to that successfully used by the railroad companies. Yet the effect of relative length of the reduction of highway mileage may be of sufficient importance to warrant some additional study. It requires no great imagination to see that the saving of 1 000 ft on a 25-mile trip would have less effect on motoring costs than the vagaries of wind and weather. As every one knows, a stiff head wind increases gasoline consumption considerably; but as yet, no one has suggested that highways be enclosed to prevent these head winds even although it may be proved, on the soundest of economic bases, that such a step would be justified in many cases.

In the foregoing, the writer has attempted to "iron out" one of the difficulties confronting the present-day highway economist. These difficulties are legion, but they cannot be solved by over-estimating the costs of vehicle operation. Other important and still uncertain basic elements in economic analysis also come to mind. What is the probable life of this particular piece of pavement? What will the maintenance charges be? What will the average traffic volume be? With what savings to the motorists may one credit the pavement during its life? These are some of the questions that must be settled. In time, engineers may be able to accumulate sufficient data to permit their successful solution. It is doubtful, however, if highway economics will ever become sufficiently exact to permit its application in anything but the broader aspects of highway planning. Within certain broad limits, highway economics will be useful in justifying certain projects. These justifications, however, will continue to be swayed by the dictates of politics, safety, preservation of natural beauty, and improvement of recreational or educational facilities.

L. W. Mahone,[17] Assoc. M. Am. Soc. C. E. (by letter).[17a]—Quoting Mr. McNeil (see heading, "Remedies for Delays Due to Marginal Interference"): "It is an admitted fact, substantiated by the Courts, that streets are primarily for the purpose of the moving vehicle." It is encouraging to know that the Courts realize this, as certainly the unthinking "public" does not seem to. It would be interesting to know how many actual studies have been made of the relative cost of providing streets for parking, as against storage lots or garages for the same number of cars.

Project after project of street widening has been undertaken, presumably to provide more traffic lanes, but actually to permit Mr. Public (to say nothing of Mrs. Public) to continue his "squatter's rights" on a section of public street, and thus maintain the usual two lanes of parked cars.

Using words of Mr. McNeil: "Marginal interference is actually created by the granting of conveniences to a few at the expense of a majority." That is exactly what happens when parking is permitted on a heavily traveled street. In the future, engineers should consider the economy of providing off-street parking for the relatively few cars that can park on the street; and thus gain two traffic lanes without widening expense. Admittedly, each case will be peculiar unto itself.

Traffic engineers realize that in heavy traffic one parked car blocks a traffic lane just as thoroughly as several. For this reason a car stopping only to unload is undesirable. As a remedy for this could not loading and unloading be taken "around the corner" and off the main thoroughfare?

In view of the various causes of traffic delay it would seem that the traffic engineer's ideal would be a street carrying no street cars or buses, with no parking, and with "around-the-corner" loading zones. Possibly, some one may be able to "put it across." The extra width of street due to the elimination of two parking lanes would in itself stop most "jay-walking."

As regards the loading and unloading of trucks in congested traffic areas, there would seem to be only two alternatives; either handle this during the night, or provide suitable space on private property. Such regulations would probably be no more drastic than many others.

A very pertinent sentence in the paper by Mr. McNeil (see heading, "Other Remedies for Delays") is: "The engineer of a municipality should strive to improve mass transportation facilities, which in the end will decrease the use of the private automobile in the busy areas, and thus reduce congestion." Much could be done to show that there is economy and time saving in using a good system of mass transportation. By placing further limitations on parking this worthy cause could be helped. The down-town merchant should be shown that he has no cause for complaint, because many are already turning to suburban stores because of traffic difficulties. He had better face the facts and "get on the band wagon" to promote the use of mass transportation.

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[17] Asst. Prof., Eng. Extension Service, Iowa State Coll., Ames, Iowa.

[17a] Received by the Secretary December 9, 1937.

Fred Lavis,[18] M. Am. Soc. C. E. (by letter).[18a]—The several papers included in this Symposium are all valuable additions to current information on this vitally important subject.

Referring to highways carrying large volumes of traffic, it is evident from the data presented that speeds of about 50 miles per hr are the maximum which should govern highway design, this being indicated both on account of safety and by economics. This also agrees with the conclusions of the writer as set forth in a paper[19] presented in 1937 at a meeting of the Highway Research Board, in Washington, D. C.

Mr. Vey's general statement that highway improvements might do away with 75% of the accidents, which now occur, does not seem to be borne out by the facts presented. Specific observations of the actions of drivers approaching certain grade crossings in Michigan, where danger signals were displayed, confirm the observations at Pittsburgh as recorded by Mr. McIntyre in the text following Table 4 and by Mr. McNeil under the heading "Remedies for Delays Due to Cross-Interference." The human element is the most important factor in the problem of highway safety, not the highway itself.

Mr. Vey's records which confirm other observations indicate that speed is an important factor both in the number and severity of accidents and, while the elimination of some of the points at which accidents occur may do away with these particular types of accidents, it seems likely that the human element, being what it is, coupled with the inducement to high speed due to the removal of some obstructions, will introduce other and perhaps worse hazards.

The improvements mentioned by Mr. Vey are desirable, because they will tend to increase traffic capacity and also because they will eliminate some causes of accidents. It seems very doubtful, however, even if the money was available to do all these things, that the accident toll would be reduced by anywhere near 75 per cent.

This latter statement has been widely quoted but to the writer it seems a very dangerous one to make in such an apparently authoritative manner as it tends to divert attention from the real cause of most accidents, the careless and incompetent driver. Mr. Vey himself states that the highway user (the driver and pedestrian) is responsible for the greatest number of accidents and admits that the money is not available to make all highways meet the highest engineering standards.

Mr. Vey's records of "before and after" results of improvements are, as he states, hardly sufficient to warrant anything like definite conclusions, but they do at least indicate that his optimism as to reduction of accidents by reason of such improvements, if the money were available, is not quite justified.

In regard to the economics of location, Mr. Schmidt's conclusions do not vary greatly from those stated by the writer in 1930.[20]

The writer cannot altogether agree with the definition of "traffic capacity" to which reference is made by Mr. Schmidt under the heading, "Physical

[18] Cons. Engr., New York, N. Y.

[18a] Received by the Secretary December 21, 1937.

[19] "Safety and Speeds as Affecting Highway Design," Washington, December 3, 1937.

[20] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 1020.

Characteristics: Widths of Roadways." The validity of this definition depends on the cost of providing additional facilities for the critical speed and the value of the time saved. As in the case of most highway traffic problems, because of the very great variety of vehicles and circumstances, this problem cannot usually be disposed of so easily.

As there is little chance of sufficient money being provided to meet and overcome present and anticipated congestion, probably some sacrifice of speed will have to be made in order to provide facilities for the passage of the largest number of vehicles at such corresponding speed.

The direct assertion by Mr. Schmidt that if increased capacity is required over that provided by a two-lane highway, the change should be to a four-lane highway, is not borne out by considerable practical experience with the satisfactory operation of three-lane highways.

The increase of distance by the use of curvature does not "obviously" (see heading, "Examples"), or necessarily increase operating expenses; the alternative might be increased rise and fall. In certain types of rolling country a judicious use of light curvature may not only decrease costs of construction, but may tend to provide a more sightly highway and one which may be more pleasing and perhaps easier to drive over. Long tangents slashing across rolling to hilly country may not provide the best location if all proper factors are taken into consideration.

HUGH A. KELLY,[21] ASSOC. M. AM. SOC. C. E. (by letter).[21a]—In any paper of the same length, the writer has never seen the entire subject so tersely, yet so thoroughly, covered as in that by Mr. Vey.

In 1936, 36 000 souls perished by automobiles! why this annual slaughter?

In 1937, 40 000 souls perished by automobiles! why this annual increase in slaughter?

Any one who has ever stood for hours watching a parade swing by to the tunes of many blaring brass bands, will have some conception of the vastness of these numbers of deaths; 40 000 living men could not pass a given point, except "at the double," and in the very closest of formations, in less than 4 hr.

Why This Annual Slaughter?—The Engineering Profession has done much to make highway travel safe in the way of providing "fool-proof" roads and safeguards of all kinds. Daily, week-end, and holiday head lines serve to prove that the last word in safe road construction is not the solution of the problem, confronting the nation, of curbing death and injury by automobile.

Mr. Vey correctly concludes: "Because of human frailties * * *, safety on the highway is extremely difficult." However, with his other conclusion—"the impossibility of supervising and governing the actions of all persons at all times"—the writer is forced to disagree, for the following reasons:

Many years ago, he attended a meeting of the Society for the purpose of discussing the proposed Holland Tunnel. During the discussion he suggested that a physical barrier be erected in each tube of the tunnel to provide separate traffic lanes and prevent cars from crossing from one lane to the other. After

[21] Civ. Engr.-Archt.-Appraiser, Jersey City, N. J.

[21a] Received by the Secretary January 8, 1938.

thorough discussion and exhaustive research, the late Clifford M. Holland, M. Am. Soc. C. E., decided that it would not be practical to use a physical barrier; that the same measure of safety could probably be obtained by installing a painted white line down the center of each tube of the tunnel and by thoroughly patrolling each tube to see that drivers kept in line and did not cross the white line.

That Mr. Holland accomplished the results by adequate patrol (the writer had this in mind when he suggested a physical barrier), is amply borne out by ten years of safe operation, during which time, and in the face of the heaviest and most concentrated traffic in the world, there have been only two accidents involving patrons, causing three deaths, the more serious of the two accidents being caused by the illegal crossing of the white line.

Why This Annual Slaughter?—When the Pulaski Skyway, the main high-speed westward traffic artery to and from Jersey City, N. J., was first opened to traffic, trucks as well as pleasure cars were allowed to use it, with the result that the Skyway, erected at enormous cost to relieve congestion and speed up traffic in the Metropolitan Zone, became known as "Death Highway." Fatalities resulting from the careless use of the Skyway, mounted to enormous proportions. Like the Holland Tunnel traffic problems, the hazards of the Pulaski Skyway, too, were solved. The Mayor of Jersey City and the Governor of New Jersey joined forces to prohibit trucks from using the Pulaski Skyway; and Jersey City furnished adequate police officers and equipment to patrol this vital stretch of costly highway properly. Fatalities ceased, and minor accidents were reduced to a minimum.

Why This Annual Slaughter?—Is it not plain, therefore, that one of the major factors in reducing deaths and accidents on the highways—one tried and found to be successful—is proper and adequate patrol of highways?

This apparently simple solution of such a grave problem has a sound psychological basis; that is, the vast majority of drivers will, to the very best of their ability, do what is right and proper, and the malicious or careless minority will fall in line when they are positively certain, as they will be under a patrol plan, that they are being watched carefully, and will positively feel the heavy hand of the law for failure to comply strictly with reasonable traffic rules.

Therefore, the writer proposes that a highway patrol be organized, that the highways be patroled 24 hr per day, that a man be assigned to such areas as he can control—which in congested districts may be not more than a half mile. A distinctive uniform to be worn at all times while on duty, and a flood-lighted car or motorcycle at night, should be provided, so that law-abiding citizens and others shall be under no misapprehension as to the penalty for failure to observe the rules of the road.

Another matter that has not been given the attention it should receive, is one that the writer presented to the State Highway Department of New Jersey many years ago; that is, to eliminate the great hazards of night driving by properly illuminating the highways.

Why This Annual Slaughter?—Motorists, of all citizens, are the taxpayers par excellence, and the public should be selfish enough to keep them alive and

free of injury for their continued ability to pay taxes, if for no other reason. The writer feels that no increase in taxation is needed to give effect to this proposed plan of proper and adequate highway patrol. Ample funds are now provided by the car-owning and driving taxpayers *via* gasoline taxes and license fees to take care of the increased personnel necessary to do away with this wanton destruction of life and property.

This is the only manner in which this blight may be removed forever from the highways of the United States and this is a solution far simpler than the sending of careless drivers to morgues, and the reading or memorizing of "horror" tales based on destruction of human life by automobiles, and other sincere but vain efforts to bring about the same result.

Why This Annual Slaughter?

SIDNEY J. WILLIAMS,[22] M. AM. SOC. C. E. (by letter).[22a]—In commenting on this Symposium, in general, the writer can only commend the thorough and sound analyses of the various papers presented. More States and cities should make these accurate detailed studies of how traffic moves and how accidents occur. Such data are the only sound bases for efforts, both national and local, to reduce accidents and congestion.

One subject, briefly alluded to in these papers, deserves further discussion, namely, pedestrian hazards, pedestrian protection, and pedestrian control. The streets and highways, traffic control equipment, and traffic control by police have been devised almost entirely to expedite and protect vehicular traffic. About 40% of the persons killed in motor vehicle accidents in 1937 were pedestrians. In cities alone the percentage is about 67, and, in large cities, it is even greater. Pedestrian deaths in cities have been reduced slightly since 1930, but have increased 64% in the rural districts.

How is this problem being met? Many persons advocate laws requiring the pedestrian to observe traffic signals, although, with a few notable exceptions, the signals were not designed for that purpose and take no account of the speed limitations of the pedestrian. They try to "educate" pedestrians chiefly by telling them to keep out of the way of vehicles, to cross at cross-walks, and with the green light, but with little corresponding "education" of the drivers to give the pedestrian "a break" when he does try to cross at the corner and with the green light.

The City of Pittsburgh has done far more for the pedestrian than most cities, yet even Mr. McNeil, in referring strictly to his topic of relieving traffic concentration, cites the considerable delays to vehicles caused by pedestrians, but mentions nothing of the enormous delays to pedestrians caused by vehicles. He suggests only "rigid enforcement of the laws eliminating jaywalking and confining pedestrian crossings to the intersections." This proposal is entirely correct, but with it must go equally rigid enforcement of the laws that give the pedestrian the right of way over the vehicle if the former is crossing at an intersection where there is no traffic control, or if he is crossing or has started to cross with a green light.

[22] Director, Public Safety Div., National Safety Council, Chicago, Ill.

[22a] Received by the Secretary January 10, 1938.

Actually, most vehicles turning on a green light do not yield any right of way to the pedestrian crossing on the same light, and most vehicles start instantly on the green light (if not before) with no regard for pedestrians who have started to cross and are thus left stranded in the middle of the street. Is it any wonder that under these circumstances many pedestrians feel they can place no reliance on the "protection" of the law or the courtesy of the driver and are thus tempted to depend entirely on their agility and dash across wherever and whenever they can see an opening in the traffic?

The writer does not intend at all to deny that a great many pedestrians are inexcusably careless and that all efforts to educate them out of this habit must be continued. He maintains only that, along with this, the engineer and the police officer must give much greater consideration to the pedestrian than most of them have done in the past. The engineer can help the pedestrian by providing raised loading platforms at street-car stops, and other safety islands in the center of wide or very busy thoroughfares, adequate sidewalks along rural highways traveled by pedestrians, and special "walk" signals where the volume justifies. Police departments can give more attention to requiring drivers to observe the legal and moral rights of pedestrians at crossings, as well as dissuading the latter from careless practices and in extreme cases arresting them. Educators, especially of the adult public, can study the entire problem more closely and can apply "selective education," which means determining the social and geographic groups that suffer the most pedestrian accidents and reaching them through appropriate channels.

The problem is well summarized by the following quotation:[23]

"In approaching the pedestrian problem it ought to be recognized that, except in a few cases of suicide, pedestrians do not want to be injured or killed in traffic. It should also be admitted that drivers do not want to be injured or killed in traffic. It should also be admitted that drivers do not want to injure pedestrians. Accidents occur where unexpected conditions arise, or traffic becomes so complex that normal walking habits and ordinary precautions do not supply the necessary protection."

[23] Rept. of the Committee on Pedestrian Control and Protection, National Safety Council, 1937.

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DISCUSSIONS

GRAPHICAL REPRESENTATION OF THE MECHANICAL ANALYSES OF SOILS

Discussion

BY MESSRS. DONALD M. BURMISTER, AND A. J. WEINIG

DONALD M. BURMISTER,[7] Assoc. M. AM. SOC. C. E. (by letter).[7a]—The purpose of the mechanical analysis is two-fold: First, to determine and record the distribution of grain sizes of a soil; and, second, to interpret the results into terms of soil character. The author has given an excellent summary of the commonly recognized classifications of soil fractions or separates. The classification based on the log 2 divisions, which the author proposes, has certain decided advantages, because it is a continuation of the familiar Tyler sieve series.

In order to interpret the results of the grading analysis into terms of soil character, the size characteristics should be described in such a manner as not only to reveal the characteristic features of the soil, but also to yield practical information in relation to general soil behavior. Mr. Campbell has stated that the tri-linear chart, used by the U. S. Bureau of Public Roads, is a convenient one for the approximate classification of soils as to texture.

The most interesting and significant statement in the paper is that the shape of the grading curve is often an indication of whether the soil is of residual, alluvial, or æolian origin, and if it is found to be residual, the curve indicates the degree of maturity. The writer proposes to carry this significant and important fact to its logical conclusion in this discussion, and, therefore, wishes to present a line of reasoning[8] different from that usually followed in evaluating the grading analysis in terms of numerical values.

Although arbitrary definitions of the mean grain size are often convenient, such as the 50% size suggested by the author, or the familiar 10% effective size described by the late Allen Hazen, M. Am. Soc. C. E.,[9] yet such definitions

NOTE.—The paper by Frank B. Campbell, Assoc. M. Am. Soc. C. E., was published in December, 1937, *Proceedings*. This discussion is printed in *Proceedings* in order to bring the views expressed before all members for further discussion of the paper.

[7] Asst. Prof. of Civ. Eng., Columbia Univ., New York, N. Y.

[7a] Received by the Secretary January 12, 1938.

[8] In "A Study of the Physical Characteristics of Soils—with Special Reference to Earth Structures." (Not published.)

[9] Annual Repts., Massachusetts State Board of Health, 1892.

must represent only temporary adaptations, which cannot be truly representative of such a diverse material as soil. A simple consideration of statistical

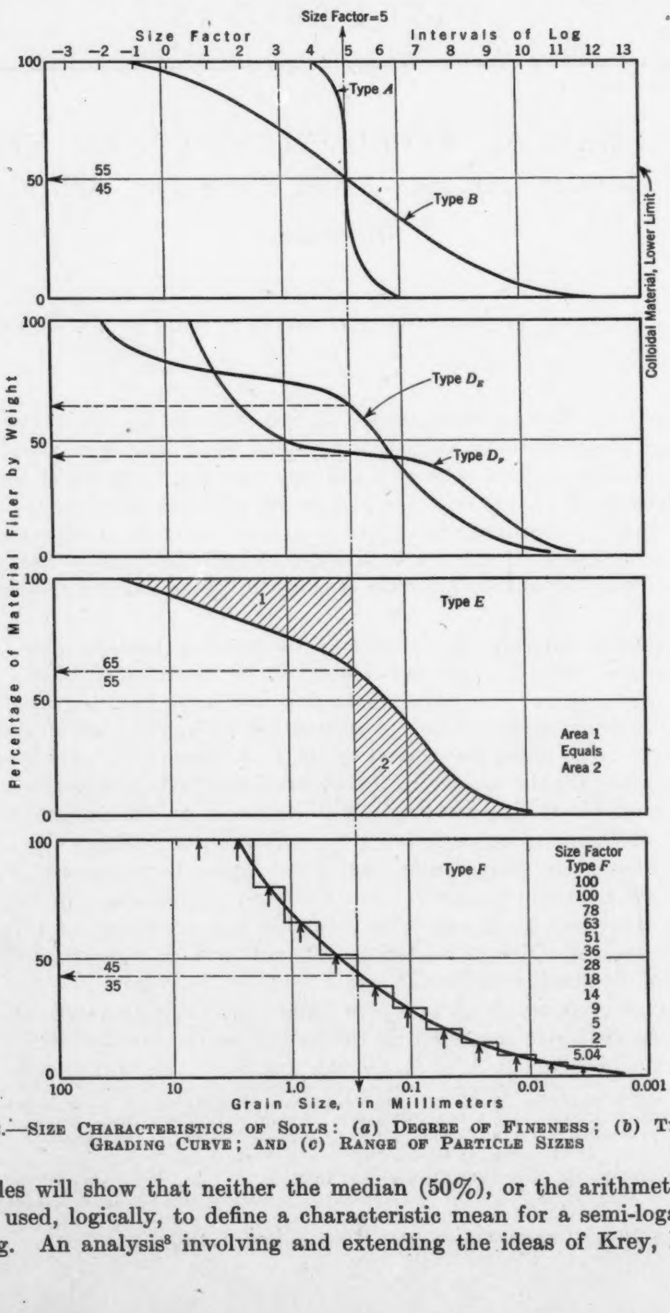

FIG. 2.—SIZE CHARACTERISTICS OF SOILS: (*a*) DEGREE OF FINENESS; (*b*) TYPE OF GRADING CURVE; AND (*c*) RANGE OF PARTICLE SIZES

principles will show that neither the median (50%), or the arithmetic mean can be used, logically, to define a characteristic mean for a semi-logarithmic plotting. An analysis[8] involving and extending the ideas of Krey, Kozeny,

and Abrams shows that the degree of fineness of the soil may be expressed adequately by a mean grain size that is defined by the length of the equivalent rectangular area which is easily obtained by means of a planimeter or by simple summation of elementary areas, as shown in Fig. 2. This mean, which is the logical one to use for a semi-logarithmic plotting, takes the entire curve into account, and is actually a weighted geometric mean. It may be shown readily, that the mean grain size is also that having the geometric mean surface area. The most significant fact about the grading curve is that the area under it is a logarithmic function of the specific surface area of the soil analyzed. The importance of surface area, as particle size decreases, is recognized in the chemical and metallurgical industries, and practical use of this idea is made in the Gates "crushing surface diagram"[10] in ore-crushing operations.

It is further convenient to express the degree of fineness in terms of a size factor, as shown in Fig. 2, which is easily computed from the grading curve, the origin of this size factor designation of log 2 intervals being taken at the $\frac{3}{8}$-in. sieve, where there seems to be definite change in the capillary tendencies of soils. An advantage of the size-factor method is that the size factors of mixtures of soils may be easily obtained by simple proportion, the size factor as a logarithm being additive, and hence the mean size can be computed.

A description of grading curves as to type is proposed by the writer, based on statistical considerations. Not only does the shape of the curve give an indication of the geological origin of the soil, as the author suggests, but it also has a greater significance, because it furnishes a basis for correctly interpreting the important influence of the distribution of grain sizes on the density of soils, whether in the natural state, or placed in some artificial state in embankments. The classification as to type of grading curve is as follows (see Fig. 2):

Type A.—Single-size, narrow-range materials, or uniform materials are represented by a nearly vertical line on the grading diagram (these materials are always the least dense and can be compacted very little);

Type B.—The normal statistical distribution of particle sizes has a nearly symmetrical **S**-shaped curve (such soils can be compacted to quite a dense state);

Type C.—This is a straight-line distribution with equal proportions of all sizes (this is a poor grading; in general, less dense than Type B);

Type D.—This type is a mixture of two fractions, one fine and the other coarse (such materials have a decided hump in the grading curve; Type D_E is a mixture of some weathered or unweathered rock fragments in a matrix of much fine silty or clayey soil; and Type D_F is a mixture of much gravel and sand in a matrix of some very fine silty or clayey soil; these materials are typical of the so-called boulder clay of glaciated regions; they pack well and may be very dense);

Type E.—In this type the grading curve is skewed toward the fine fractions (that is, it is predominantly fine with some coarse material; the density is determined by the character of the fine material and is usually less than that of Type *B*); and,

[10] "The Handbook of Ore Dressing," by Taggart, p. 496.

Type F.—The grading curve in Type *F* is skewed toward the coarse fractions and is predominantly coarse with some fine material. This type approaches the ideal grading curve for maximum density (the Talbot grading) and conforms to the requirements for the best stabilized earth road mixtures of the U. S. Bureau of Public Roads. Type *F* material in its loosest state may have a greater unit weight than any of the other types in their most dense states.

An interesting and significant fact is that the percentage corresponding to the mean size is not now an arbitrary constant quantity, but varies in a characteristic fashion with the type of grading curve, and, in reality, represents a skew factor. In Types *A*, *B*, and *C* the percentage may vary from about 45 to 55, and the median approximately coincides with the geometric mean. Since Types *E* and D_E are skewed to the fines, the percentage corresponding to the mean is considerably greater than 50, usually from about 55 to 65. In contrast, the percentage for Types *F* and D_F is considerably less than 50, usually from 35 to 45.

In an attempt to describe the grading curve by the slope of the curve or any part of it, there has been a confusion of two separate and distinctly independent ideas—the shape of the curve and the range of particle sizes—which cannot be expressed by any single factor. With degree of fineness defined by a geometric mean size or size factor, and the shape of the curve defined by the type, the size characteristics are finally defined simply in terms of the range of particle sizes, with no reference to slope, as such. The designation of range to take in the entire grading curve may be expressed conveniently as the ratio of the 95% size to the 5% size, or by the number of size factor units (intervals of log 2) between these two extreme sizes. For practical purposes the lower limit of the designation is taken at 0.001 mm and all materials finer are considered as colloidal materials.

The range of particle size also has a most important affect on density. For the same type of grading curve the greatest density is found in the material having the greatest range of particle sizes, and the least dense is always the narrow range Type *A* material.

On these bases it seems logical to define the size characteristics of soil in terms of the three independent factors, which not only have statistical but also physical significance in the grading-density relations,[11] and to define the soil fractions or soil separates in terms of a log 2 series, which is a continuation of the Tyler sieves.

A. J. Weinig,[12] Esq. (by letter).[12a]—The comparison of mechanical analyses and nomenclature of soils offered in this paper is particularly valuable in view of the many different systems included, and Mr. Campbell's proposed plans would seem to be a logical, general outcome of all these classifications.

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[11] See p. 364.

[12] Director, Experimental Ore Dressing and Metallurgical Plant, Colorado School of Mines, Golden, Colo.

[12a] Received by the Secretary January 14, 1938.

In the matter of diameter-grade designation, a suggestion is made that if the type of distribution curve be represented by a letter, an instant mental picture of the soil type would be had. Thus, for example, if (C) represents convex; (K), concave; (S), S-type n-curve; and (L), linear, then the diameter-grade designation, of the samples given in the paper, for example, would be:

Sample:	Diameter-grade designation
(*a*)	D-0.0038–G-1.9 C
(*c*)	D-0.081 –G-1.0 S
(*e*)	D-1.45 –G-3.3 K

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DISCUSSIONS

ECONOMIC ASPECTS OF ENERGY GENERATION A SYMPOSIUM

Discussion

By Messrs. W. S. Finlay, Jr., R. M. Riegel, Ralph Bennett, J. M. Mousson, Ben C. Sprague, V. M. Marquis, R. L. Sackett, D. J. McCormack, and I. E. Moultrop

W. S. Finlay, Jr.,[28] Esq. (by letter).[28a]—In any discussion of the economic aspects of energy generation, one can distinguish at least two levels—the level of pecuniary analysis and the level of social or human analysis. However, one is scarcely justified in so limiting the range of vital factors dominating the life, opportunity, and progress of the power industry.

For instance, there is a certain type of professional mind which has been identified with much that has made the industry what it is, and whose contributions are of essential importance, but which is motivated solely by the compelling thought of producing something of a highly technical order, without regard to the bearing their product may have on economic, social, or human aspects. That sort of mind can be classified as similar in its make-up to that of an artist who endeavors to reflect his ideals on the canvas upon which he paints.

Such a contribution is certainly worthy of consideration as occupying a third level. True, economics may enter into work on such a level, but history shows that it is frequently only in the last stages of development that the vision of an inventor's mind is forced into a mould that relates itself to economics. The incandescent lamp was long in existence before it was given commercial value by Edison and adapted by him to human use and service.

It may thus be possible to isolate within other levels some of the vast range of activities that compose the power industry, but is not the isolation of components working in a contrary direction to the purpose of a symposium? Is

Note.—The Symposium on Economic Aspects of Energy Generation was presented at the meeting of the Society and at the Joint Meeting of the Power and Engineering-Economics and Finance Divisions, Pittsburgh, Pa., October 14, 1936, and published in the December, 1937, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the Symposium.

[28] American Water Works & Electric Co., New York, N. Y.

[28a] Received by the Secretary October 14, 1936.

not such a purpose that of discovering, if possible, a reason for the theory that the general activities of the power industry, involved in the generation and use of electricity, cannot be isolated one from the other in levels, or in any other way? Is it not a fact that in any industry the viewpoint of one who would appraise it must be that which embraces it as a harmonious whole of men, machinery, money, materials, and more?

With the background of such an argument, the writer assumes, for purposes of discussion, that this particular form of industrial activity is indivisible into any component parts. There is a constant necessity for "give and take" among its pecuniary, social, sheerly technical, and political components. This Symposium supplies a background in indicating only a few of these interrelationships. To bind the parts together into a harmonious whole, and in a manner thoroughly satisfactory to all who have interested themselves therein, particularly in these latter days, apparently calls for a master mind beyond any yet revealed. At least, no one in the United States in whom students of the problem might have implicit confidence has supplied anything like an answer—anything that might direct the trends of effort and research, governmental or industrial, toward a single, ultimate, and ideal objective. One is impressed rather with a present tendency to tear apart in every direction—the belief on the one hand that the object of governmental authority is to destroy, and on the other hand that it supplies the only means by which to advance.

Concentrating Objectives.—The titles of the papers in this Symposium typify in a small way the diversity of interests that motivate many minds whose theoretical objective is that of a common purpose. Long before the use of electric energy became a factor of importance in living, Man struggled to maintain life, to accumulate wealth, and thereby to promote the comfort of living. As incidentals thereto he adapted machinery to his purposes, he endeavored to transform energy from one form to another in aid of such purposes, he regimented other men in his service and, in turn, adapted his own activities in relationship with those of other individuals, groups, and nations. The whole created a civilization. Electricity entered, became a force, an element of value, a service, a means for creating wealth, and grew almost over night into an element of major economic importance, theoretically affecting human welfare not only here or there but almost universally. Certainly, so far as the United States is concerned, it has been elevated to a pedestal of national importance. Whether or not such importance is exaggerated is a matter open to discussion. Unquestionably, from the angles of politics and publicity, it is occupying, at present, rather a top position. Perhaps it is a proper part of this discussion to inject the throught that this position of importance has been exaggerated to a degree for which there exists no warrant, social or economic.

Near the beginning of his paper, Mr. Freeman states: "It is hardly necessary to dwell upon what is the most obvious result of cheaper power, namely, increased well-being." In that sentence he places his finger upon the theoretical yeast that has inflated an enormous loaf of political aspirations and popular theories that at best have an uncertain background of factual warrant. If it were not for the popular idea that an unlimited amount of electricity is a sort of

Aladdin's lamp for the mere rubbing of which any individual can find relief from labor, from care, from expenditure, there would be little popular support to the political plaything that electricity has come to be. The importance and value of Mr. Freeman's key sentence depend wholly upon whether electric power can be cheap, and, thereafter, upon what contribution electric power can make to economic well-being. Without pursuing the theory to a conclusion, certain facts drawn in part from the other papers of the Symposium can be introduced to indicate the doubt of much of the social theory that supplies the background for such a key sentence. It is the writer's purpose to examine this factual background and to the extent to which the key sentence is affected by such an examination, the entire discussion is affected.

The writer may be one of a minority who holds a definite belief that the social importance of what is termed "cheap power" has been greatly exaggerated—in fact, exaggerated to such a regrettable extent as to create a demand for something that cannot be supplied. For this reason, there is bound to result deep disappointment upon the part of many, with the result that the social aspects of the general problem will take the form of resentment against those who made promises. For instance, were one honestly to examine into the net social effect of electricity as measured by the distribution of population, it might be found to be rather minor except as it is affected by and related to improvements in forms of transportation facilities.

The generation of power in bulk in certain favorably located centers reflects only a certain and peculiar set of conditions. Mr. Sporn emphasizes the economic limitations imposed upon power distribution from certain centers by his comparisons to the cost of transporting fuel for the generation of such power. Certain limited hydro-electric sources of power create special situations; but others too often tend to concentrate population in industrial areas about a source of energy, thus contributing to the difficulties resulting from such a concentration rather than supplying a solution to the problem.

To certain realistic minds, the social effects and value of electric energy supplied from a central source and distributed to the farm and rural home are distinctly uncertain and untried. To the actual costs of such service must be added the cost of electrical equipment, the fixed charges on which almost unquestionably exceed materially the actual price of electric current. The ability of the farm to carry the burden of cost will depend directly upon its producing capacity and earning ability in ways that are wholly unrelated to the availability of central station electricity to the farm.

Carrying the thought a bit further, the farmer may actually be impoverished by his attempt to carry the burden of the actual cost of such electricity. In other words, he might better continue the use of other and cheaper forms of energy plus possibly such an adjunct as an oil-engine generator set for home lighting and other minor service.

As relating simply to its social effect and value, of course, electricity in the home has a certain place of importance as a substitute for gas and oil, but there is nothing in that importance which calls for a complete re-organization of the service background of electricity as a commodity. In other words, there is no peculiar social value in service and power generated by a publicly owned central

station that is not inherent in similar service and power supplied by a privately financed and operated source. In fact, the development and progress of any form of public service, be it electricity, transportation, or any thing else, have been far greater under the spur of private initiative than under governmental ownership and direction. Practically, every feature of value in power generation and service in nearly every publicly owned and operated plant in the United States had its origin under the stimulation of private investment and initiative.

In brief, the writer can find no warrant for attaching extraordinary social values to the manner in which electricity is generated and used. Yet these values have been seized upon as the basis for wholly illogical argument and masses of propaganda designed to transfer the industry in part and eventually as a whole to public ownership and operation. History has proved that under public regulation and private management, transportation plays as great a part as power—if not, indeed, a greater part—in the social life of the nation. Again, the production and distribution of food have equal, if not greater, social significance. Everything that relates to life, to ease and comfort, in general, has no more and no less social significance, and no one factor should be singled out for special treatment, control, or direction.

This general concept has been recognized for years by the man who has specialized in electricity and electrical service—that is, by the power producer, who relates every phase of his service, from the financing stage onward, to a consideration of its proper use at the point of consumption. In brief, he is the privately owned power industry of to-day.

The Technical Aspects of the Power Problem.—The various papers which have set forth individually the technical engineering features of the problem are inter-related and might almost have been combined in a unified whole. Progress in the art of power generation, as described by Mr. Orrok, is a magnificent record of achievement upon the part of a record-making and record-breaking industry; and is clearly indicative of extraordinary possibilities for the future, provided nothing is done to stifle private initiative and inventive genius. Mr. Rogers' paper illustrates the extent to which specialization in one field of power generation tends to contribute to a well-organized and forward moving whole. As an addendum, a paper might have been prepared, setting forth the extraordinarily complicated combination of the various forms of engineering comprised in a modern steam station. Thereto might have been added the complex problems of an electrical and economic type that the engineer is called upon to organize in making up a properly balanced power generating, transmitting, and distributing system.

Economics in Engineering, Generation, and Distribution of Power.—Mr. Justin has touched briefly upon some of the "high spots" of the economics of power transmission and distribution. The brevity of his presentation is possibly responsible for its apparently being rather more idealistic than practical in its concept. For instance, inter-connection of systems with the objective in mind of securing some advantage of an economic character is, and for years has been, merely a part of the ordinary day's work of the electric power industrialist. The value of these inter-connections may be based upon any-

thing from economies in financing to insuring reliability of service to some remote customer. The history of the industry has been notable for the extent to which advantage has been taken by the various industrial units comprising the industry, of co-operative action to procure results of maximum economic interest, not only—as has been frequently over-emphasized—that accruing to the stockholder, but to the user, whether a large industry or an individual whose interest is limited to the service in his home. This fact is not as generally appreciated as it should be, and those who are prompted by a motive which might result in some form of prestige have deliberately endeavored to obscure the fact. On its part, the power industry has been backward in advertising.

The best evidence of the fact that the power industry has taken advantage to an extraordinary degree of every opportunity to cheapen and stabilize the service is the rate of progress that it has made in a very brief span of time. No industry has made, or is making, a like rate of progress.

Mr. Sporn's exceedingly well-written paper is based upon facts and figures rather than on theory. However, he cannot resist the temptation to which all engineers are subject, namely, to do a bit of forecasting. He refers in particular to probable trends of various departments of service within the industry and of economics relating particularly to production with the consequent effects on costs of the service.

There is this in favor of the reasonableness of Mr. Sporn's forecasts: The power industry of to-day is the result of years of intensive combined effort on the part of those engaged therein, which is in no way limited to engineering alone but involves everything that goes to make up a progressive industrial service—financing, sales methods and psychology, social effects, technical engineering, the perfecting of mechanical processes, etc. Unlike the average industry, it is not limited in its advancement by the advantage gained by individual units from the development of patented or special processes. Its extraordinary progress has in the main been due to the readiness—the actual anxiety—of those engaged in it to exchange ideas and information as to every phase of performance or detail of service.

This type of progress inclines one to the opinion that little or nothing has been overlooked in the way of developments that have contributed to the success of the industry, its efficiency, or performance. There are no secrets—no developments that are not fully known or made use of if needed or wanted. The industry is one which has led and directed the manufacturer in his supply of its tools of service. True, the manufacturer in his own laboratories has devised many a type of equipment of value to the industry; but the greatest laboratory has been the field of service—and the directors in that laboratory have been the power industrialists themselves, with the manufacturer as a most co-operative and able partner.

In the light of the foregoing, it is rather difficult to disassociate one's ideas as to future trends from what has been and is being accomplished. It would seem illogical to think that any sudden discovery will eliminate the great waste of heat energy that now attends the generation of electricity in steam plants; hence, one's ideas as to the obsolescence of generating machinery are developed rather in terms of something progressive than in the direction of

something revolutionary that might suddenly destroy vast values in equipment, transmission lines, etc.

Nevertheless, there are those who envision the possibilities of new and wholly unknown ways of energy transformation and concentration adaptable to the service of mankind; however, if there is any precedent in history, it is that such changes can only be gradual and as the result of persistent, progressive, time-taking invention and research.

If that theory as to the future of the industry is correct, there is every reason for maintaining it as a wide open field of opportunity for private initiative and inventive genius, rather than permitting it to become a political plaything, stifled by bureaucratic control and direction. Responsibility for progress in the power industry, as in every other industry, in a democracy such as that of the United States, must rest in the application of individual effort spurred by the profit and success motive, and not in the benevolent paternalism of the political appointee or aspirant. The economics of power production still remain essentially a matter of endeavor upon the part of those who would apply themselves to intensive specialization, devoting their span of life thereto. It is not and cannot be the hit-or-miss achievement of the dilettante in politics, sociology, or reform for reform's sake.

R. M. Riegel,[29] M. Am. Soc. C. E. (by letter).[29a]—The advances in the art of turbine design and manufacture which have been described by Mr. Rogers, particularly those in the field of the propeller turbine, have been very important. Special consideration should be given to the extensive use of the manufacturers' laboratories, in which so many of these advances have originated. The staffs of these laboratories conduct their work with care and scientific exactness, and their product has been extremely valuable. However, the results of model tests must be verified by experience with the prototypes, and occasionally phenomena occur in the full-sized units which have been too obscure to be observed in the small models. In studying the performance of full-sized units, the power engineer functions in furtherance of the research in the manufacturer's laboratory. Other important research has been performed by interests which purchase equipment.

The use of large propeller units is comparatively new, and some facts still remain to be learned about their characteristics. The ways in which the power engineer will contribute to the art is illustrated by a case of the TVA. The two units at the Pickwick Landing Dam were designed with a horizontal splitter in the draft-tube. The two units at Wheeler Dam have no such splitter. The units at the two plants are quite comparable in size, head, and discharge, the Pickwick units being a little larger. The splitter is a difficult and expensive detail of construction in units of this size and should not be used unless it is advantageous. From the comparative performance in these two plants the TVA and the Engineering Profession will have evidence to determine its merits.

Another investigation of the TVA indicated that it was feasible to tip upward the horizontal leg of the draft-tube in units of this kind to a degree which

[29] Head Civ. Engr., TVA, Knoxville, Tenn.

[29a] Received by the Secretary October 14, 1936.

previously had not been deemed feasible, and the specifications for the Pickwick units required that a high slope be used. In the dams of the TVA, the large propeller units require considerable rock excavation to accommodate the deep draft-tubes, and the tipping up of the horizontal leg represents a real saving in construction cost and a widening of the field of this type.

Another investigation in the laboratories of the TVA is that of a head-increasing device which may be applied to the dams on the Tennessee River. Head increasers have been developed by a number of engineers, including Tefft and Thurlow; and it may be that the proposed device can be worked out to widen the field of use of such appurtenances. It may be noted that these devices have been developed outside the manufacturers' laboratories.

The art of hydraulic generation has been advanced, furthermore, by the development in recent years of various accessories which contribute to safe and economical operation—improvements in governors, penstocks, trash rakes, gates, and similar devices. One noteworthy contribution is the "paradox gate" developed by the U. S. Bureau of Reclamation as a reserve valve for the needle-valves at Boulder Dam.[30] A modification of this type has been used for the entrance gates to the penstocks at Norris Dam. These gates, 16.5 ft by 28.5 ft in opening, are of heavy steel, and ride on caterpillar trains in a manner somewhat similar to the well-known Broome design. The seating device, however, is radically different, the gate being seated by a horizontal movement after it has been lowered into its proper position. These gates were installed in 1936, and are apparently operating satisfactorily.

In conclusion, it is worth while to note that co-operation between manufacturers and purchasers' engineers has led to numerous improvements in the past. Such continued co-operation will solve the problems of the future.

Ralph Bennett,[31] M. Am. Soc. C. E. (by letter).[31a]—It is certainly true that the efficiency of power production must increase rather slowly in the future. The great advances have already been made. Only those refinements which come with detail study of specific cases and the use of more complicated equipment are left.

One of these refinements is provision for more efficiently carrying peak loads on plants especially designed for the economical delivery of power on a very low load factor. This does not mean merely improvement in direct generating efficiency to carry short peaks. It may mean an artificial time distribution of the load within the system generating plants, so that the total power sold will have the lowest cost in fuel and labor and in capital invested.

To some extent power plants having short intermittent peaks at close intervals can store heat to cover these peaks in heat accumulators. Electricity was once stored, for peak use, on central station networks in lead storage batteries; but other methods of carrying the peak have superseded the storage battery, except in the case of small isolated plants. In some countries, as Mr. Justin has stated, the use of off-peak power to pump water to high-level

[30] *Mechanical Engineering*, October, 1934.

[31] Cons. Engr., Los Angeles, Calif.

[31a] Received by the Secretary December 13, 1937.

storages for rapid release through turbines during the station's peak, is well established. Thus, the external peak is met through a redistribution of station load within the system, fuel efficiency being sacrificed in favor of a lower total power cost.

In the typical pumped-storage plant, water is drawn from a convenient stream and returned to it after use. The reservoir is of small size. The up flow and the down flow of water are frequently through the same pipe, and the pump and the recovery turbine are commonly connected to an electrical unit which is run as either a motor or a generator. This plant is efficient under only a very limited range of conditions, but the energy thus recovered as peak power is very much more valuable than the off-peak power used in pumping the water. It is possible to recover about two-thirds of the power used.

The profitable investment in such a plant is much more than is permissible in a fuel-burning plant, since the rent of the money invested is almost the entire cost. The allowable investment may even be greater than the cost of a distant water power and a peak-load transmission therefrom, because the storage recovery plant, being closer to the point of use, will have lower transmission losses and greater reliability.

The pumped-storage peak-power plant, split into separate pumping and recovery plants connected by a high-level conduit and storage reservoir, can be used to carry a water conduit over a mountain range instead of around it or through a long tunnel beneath it. Many cities import their water through such conduits and, in many cases, the water system can be easily arranged for pumped-transit power recovery. The higher the lift the less the percentage value of the fixed losses and the greater the percentage of the pumping power recovered. If the mountain range is high and a high-level summit storage is available the system as a whole can be very efficient.

However, the conduit line used for peak-power recovery will differ materially from the classical constant-flow low-level conduits as now designed. It will need a re-arrangement of petty storages and of pumping plants such that the long conduit can flow continuously. The pumps will run on a 15 to 20-hr schedule, and the regeneration plant will make its entire recovery in only 4 to 6 hr.

If the system is operated on a day-to-day basis, the terminal and summit reservoirs need still be large enough only to impound the flow during its manipulation for peak-power delivery. However, if the summit reservoir can be large, and the terminal reservoir can be proportioned for seasonal as well as daily variations, the value of the pumped-transit system will be immeasurably enhanced. In such a system lush supplies of water and seasonal excesses of power may be combined to pump available water to the summit reservoir with secondary power and thus provide potential peak power and water for a long time ahead, independent of the fluctuating rates of the stream or of primary power production.

The use of the incoming water for power production is quite compatible with the total rate of water flow required by the city. A population unit of 5 000 will use 1 cu ft per sec of water (continuous flow), and will absorb the output of a 1 250-kw power plant.

Among the cities where this type of project would appear to be of value, four may be singled out for special mention. The situations at San Francisco, Santa Barbara, and San Diego, Calif., and at Denver, Colo., are strikingly similar. In each case, an imported water supply encounters a high mountain range close to its destination. Long low-level tunnels can be driven through these mountains at great expense, or pump-fed conduits can be carried over them and through summit reservoirs and peak-power recovery plants at relatively low cost. In each case the intervening mountains are high and the total lift correspondingly great. Such a situation is advantageous in a pumped-transit system, because the fixed losses (such as conduit friction, entry head, reservoir fluctuation, and terminal reservoir elevation) are independent of the lift. This point is illustrated in Table 5, in which the power possibilities of a low-level and a high-level route for the proposed San Diego water supply are compared.

TABLE 5.—POSSIBLE ENERGY RECOVERY FROM PROPOSED SAN DIEGO, CALIF., WATER SUPPLY

Item	Type of Route	
	Summit lift, no tunnels	Low-level conduit, with 25-mile tunnel
Lift, in feet	4 000	1 100
Conduit loss, in feet	100	120
Delivery head, in feet	550	550
Net regeneration head, in feet	3 350	430
Gross recovery, percentage	83.7	39
Efficiency of combined plants, percentage	76	76
Net recovery, percentage	63.61	29.64

In fact, an almost perfect example of the possible advantage of such a system is supplied by the proposed importation of Colorado River water into the City of San Diego across a high mountain range, on which numerous high-level reservoirs are already in use. The generalized profile across the Coast Range (Fig. 36) illustrates the facility with which this particular diversion can be adapted to pumped-storage transit.

Steam plants in San Diego are the primary source of power. The most distant regeneration plant would be only 40 miles east of the city; and the closest, only 15 miles. Power for the large pumps would be carried by the same transmission line, and for a maximum distance of 60 miles. Other advantages include the large high-level reservoirs already in use along the proposed conduit line, and the large low-level storage and distribution reservoir at the city's edge.

Elaborate calculations covering the allowable cost and probable kilowatt and kilowatt-hour outputs of a pumped-storage plant considered as a peak load plant for a large system are contained in a discussion[32] by A. H. Markwart, M. Am. Soc. C. E. His conclusions are based on a particular, but perhaps representative, set of conditions. Under a condition of practical balance between off-peak pumping and peak-power release, from 13 to 20% of the

[32] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), pp. 911-932.

system energy can be used for pumping. The regeneration plant would probably have a load factor of only 10 to 18%, and would generate only 8 to 12% of the total energy delivered to the system. Yet the peak which could be carried would be 30 to 50% of the system peak.

Under the conditions considered by Mr. Markwart, a pumped-storage plant costing from $120 to $150 per kw, may be economically feasible. However, with pumped-transit storage, part of the money saved on conduit tunnels can be properly applied to additional plant; and although the total machinery cost may be larger, the return on the money invested in the combined water-transit and peak-power system may be much greater than the net earnings of separate plants to perform the same duties.

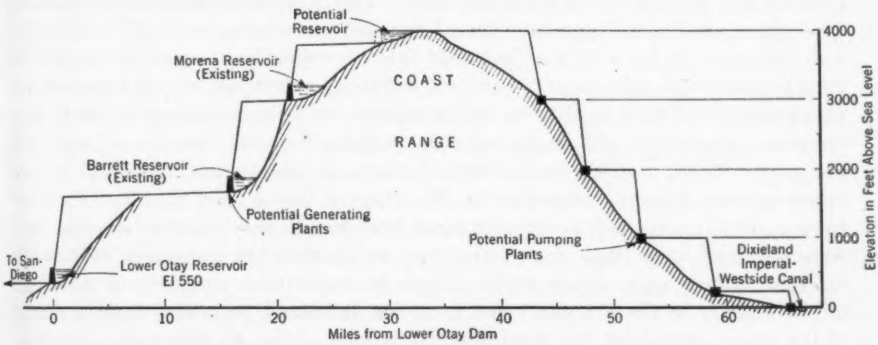

FIG. 36.—PROFILE OF PROPOSED SAN DIEGO, CALIF., WATER SUPPLY CONDUIT, SHOWING POTENTIALITIES FOR POWER RECOVERY

In conclusion, it may be stated that it is possible to design a water supply system, using pumped transit and summit storage with retarded regeneration, such that intervening heights may be made to decrease the first cost of the water and power system and to increase its earnings. In general, the higher the lift, the greater will be the net recovery; and the shorter the regeneration time, the greater will be the net earning.

J. M. MOUSSON,[33] ESQ. (by letter).[33a]—The paper presented by Mr. Rogers gives an excellent review of the progress made in hydro-electric generating equipment. Since it is often widely believed that a stage of stagnation in development of hydraulic prime movers has been reached, it is gratifying to see that Mr. Rogers points out various fields in which further improvements are yet to be expected.

He mentions, for example, the ever-increasing heads under which fixed-blade propeller and Kaplan turbines are being installed, and refers particularly to the Shannon River development, in Ireland, where a Kaplan unit is operating under a head of 106 ft. It is interesting to note that, in 1937, the Electricity Supply Board of Ireland (Dublin), decided to install Kaplan turbines at the new Liffey Power Plant, where a head of more than 130 ft will be utilized.

[33] Hydr. Engr., Safe Harbor Water Power Corp., Baltimore, Md.

[33a] Received by the Secretary January 4, 1938.

In the field of pump-turbine units substantial progress has been made. Although Mr. Rogers refers only to laboratory studies with modified high- and medium-head Francis-type runners, an actual installation of a low-head pump-turbine unit was completed early in 1933 at the Baldeney Power Development, on the Ruhr River, in Germany.[34] The unit is operating under a maximum head of about 32.5 ft. Its design was based on laboratory studies of a modified 5-bladed Kaplan runner with fixed guide-vanes. The runner is coupled directly to a 50-cycle alternating-current asynchronous machine operating at 255 and 326 rpm when generating and running as a motor, respectively. Although capacities and size are relatively small (power output, 1 100 kw; power input, 1 300 kw; runner diameter, 59 in.), this installation seems important as a pioneer in the low-head field. The maximum efficiencies reported for turbine and pump operation, based on acceptance tests, are 90% and 78%, respectively. In view of the fact that for the water measurements propeller current meters of only one type (25 cm in pitch) were used, which even under practically ideal field conditions under-register by an appreciable amount, the foregoing prototype efficiencies should, perhaps, be corrected to 88 and 80 per cent. There is little doubt that the average maximum efficiency of the pump-turbine designs referred to by Mr. Rogers, based on a modified Francis runner, may be superior, keeping in mind that Mr. Rogers' results (see Fig. 21) were obtained on a 16-in. model and that an appreciable step-up in efficiency for the Francis type, pump-turbine may be expected. Aside from possible improvements in the Kaplan type, however, it should be noted that in spite of the lower average of the maximum efficiencies of the Kaplan turbine-pump installation referred to, better over-all efficiency may perhaps be achieved with this type of design, if the unit has to generate at part load over long periods of seasonal low-river stage, as at the Baldeney Plant.

In addition to the mechanical improvements referred to by Mr. Rogers, mention may be made of the possibility of injecting air into low-head propeller-type units to reduce vibration at high-gate opening. For instance, at the Safe Harbor Plant, Safe Harbor, Pa., vibration alone was the determining factor for the gate limit under normal operating conditions. Because of the hazard involved, even short periods of over-load had to be avoided. Air injection through the head cover proved so successful in reducing vibration that more than 1 000 kw per unit, or more than 6 000 kw for the entire station, were gained in output by stepping up the gate limit without increasing the vibration above the previous intensity registered at the lower gate limit and no air injection. Air injection of 125 cu ft per min through the head cover under high loads is now a standard operating procedure at Safe Harbor.

Air injection may be used in the future not only to improve operating characteristics, but to provide a cushioning medium to render cavitation ineffective. With ever-increasing heads for propeller-type turbines, a point may be reached where even the best stainless alloy steels will no longer resist the attack from severe local cavitation. If air injection for this purpose is to be economically feasible, however, research and model tests will be essential,

[34] "Die Turbinenpumpe in Stauwerk Baldeney" ("The Turbine-Pump at the Baldeney Power Plant"), by O. Spetzler, *Zeitscrift* des Vereines Deutscher Ingenieure, Vol. 78, No. 41, October 13, 1934, pp. 1183 to 1188.

because with the large volume of air required it is necessary to make the injection in such manner that the major part of the air will find its way to the particular location desired.

BEN C. SPRAGUE,[35] ESQ. (by letter).[35a]—In evaluating the saving resulting from the reduction of new generating-capacity requirements accomplished by inter-connection of power supply systems, Mr. Justin assumes this saving to be equal to the total fixed charges and residual operating expenses in connection with the generating capacity additions that are avoided. Actually, the addition of new generating capacity to a power system results in very appreciable operating savings, allowance for which gives a net cost of providing additional generating capacity of appreciably less than the additional fixed expense involved. These savings are obtained by operating the new generating capacity to carry the base part of the system load, replacing a similar amount of capacity which otherwise would have been operated at or near the top of the load, and which normally would have a much higher operating cost than the new generating capacity. For example, if the new generating capacity has a coal rate of 1.0 lb per kw-hr and can be operated 7 000 hr per yr to replace capacity having an average coal rate of 1.7 lb per kw-hr, an annual saving of 4 900 lb of coal—which at $4.00 per ton would amount to $9.80—would be realized per kilowatt of new capacity added. Based upon a new capacity cost of $130 per kw and a fixed charge rate of 14%, the operating saving would amount to 54% of the annual fixed charges of $18.20.

As a result of the continuous improvement that has been made in power-station design throughout the past, the saving in operating cost has always been a factor to reckon with in determining the annual cost of providing additional generating capacity. This factor has had an appreciable bearing upon the economy of utilizing hydro power for carrying the peak of the system load. Because of the very low annual load factor of the load peak to which hydro power might be applied, the low operating costs obtained with new steam capacity, *per se*, have had little effect on the relative economy of steam or water-power capacity additions. Rather, it is the lower production cost of the new steam capacity, as compared with that of the capacity it could replace, which is so important a factor in steam and hydro comparisons.

Since steam station heat rates are fast approaching an irreducible minimum, it is to be expected that in the future the savings in operating cost resulting with the new steam capacity additions will be less of a factor in reducing the cost of providing additional generating capacity by steam. However, the making of superposed capacity additions has introduced a new factor into this situation. If the high-pressure boilers provided in connection with superimposed capacity additions were of the same efficiency as the low-pressure boilers they replace, the heat rate of the new capacity provided by superposing would be approximately 5 000 Btu per kw-hr. However, the new high-pressure boilers provided are usually so much higher in efficiency that the new superposed generating capacity can be loaded with an actual reduction in fuel

[35] Engr., West Penn Power Co., Pittsburgh, Pa.

[35a] Received by the Secretary January 5, 1938.

requirements. In other words, the new generating capacity has a negative coal rate. Thus, the savings in operating cost made possible by superimposing, while not sufficient to cover the entire fixed costs involved, are sufficient to result in a low net cost of providing additional steam capacity.

Because the advent of superposing has resulted in reducing the net cost of providing additional generating capacity, it has tended to make inter-connections, peak-water power, and pumped-storage installations less economical than they were previously. However, this effect is only temporary and will be followed by a reverse effect. Superposing can only be utilized as long as low-pressure capacity remains which is economically adaptable to superposition. The low-pressure plants superposed upon will be rendered almost as efficient as new condensing capacity that could be added at present. Thus, when the time comes that further superposing is no longer economical, there will be less inefficient generating capacity in operation to be replaced by the new generating capacity. Hence, the operating cost saving will be greatly reduced. As a result, conditions may become more favorable for the development of peak hydro than they have been for some time. Considering that the need for peak hydro also increases with the load peak (because of the resulting greater amount of low load-factor peak that can be economically carried by hydro), it does not seem unreasonable to expect a resumption in development of hydro capacity after the possibilities in superposing have been exhausted.

It is thought that in many cases the difficulties in properly evaluating the reliable capacity values of run-of-river hydro-electric plants have resulted in their not being given full credit. Because during a large part of the year little output can be definitely depended upon from the run-of-river plant, its peak-carrying capacity value has been regarded as little more than that resulting with the minimum stream flow. In reasoning thus, it is forgotten that steam capacity cannot be relied upon as being available at all times. It is only by providing spare units that the probability of having sufficient capacity available to carry the load can be made great enough to meet the practical requirements. Since the availability of steam capacity and even the amount of the system peak to be carried are subject to chance variations, it is proper to evaluate water power by considering its effect upon the total probability of having power available to meet system needs.

In making such evaluation, allowances should be made for the ability to schedule steam-unit outage times so as to offset the seasonal variation in hydro-electric output, and also for the ability of the run-of-river hydro to give full output for short periods at times of emergency, by drawing upon its limited storage. Where the potentialities of remaining undeveloped water powers are properly determined, it is believed that many of them will be found economical even though they are much less favorable than Niagara.

In Fig. 26, Mr. Justin shows the firm capacity of a hydro-electric plant to increase in direct proportion to the system load. Because such a rate of increase of firm capacity would require increasing amounts of controlled energy output, it will be evident that the firm capacity of a given hydro-electric development increases at a slower rate than the system load. Usually, the firm capacity of a given hydro-electric development varies approximately as

the square root of the system load, assuming, of course, that the shape of the load-duration curve remains about the same.

Mr. Sporn has made a valuable contribution in showing that it is not economical to transmit steam power over long distances. It should be stressed, however, that Mr. Sporn's comparisons are made for conditions not frequently met in actual practice. As a result, the comparisons show a much lower cost over the entire range of load factor, with transmission by freight over a distance of thirty miles, whereas it is common practice to transmit power electrically over this distance rather than resort to local generation. The failure of the comparison to correspond with what is found economical in actual practice results because it does not take account of the differing amounts of spare generating capacity required or of the necessity for inter-connection lines even when there is no normal delivery of power. Further, there is a practical necessity, in almost any case, of transmitting power for at least some distance from the generating station to the centers of distribution. Hence, if the generating station is displaced from the center of the territory supplied (in order to locate it at a coal mine, for example) the additional transmission cost involved is usually much less than the total cost of transmission over a similar distance, shown in Fig. 26.

Thus, because the actual conditions encountered are different from those assumed in Mr. Sporn's comparisons, it is frequently economical to transmit power over long distances rather than to resort to local generation involving freight delivery. Although Mr. Sporn's cost determinations necessarily had to be limited to some simple set of conditions, the variations from the conditions of actual practice should be kept in mind so as not to arrive at any wrong conclusions as to the economy of power transmission. Because Mr. Sporn's results would seem to indicate that transmission by freight is cheaper over all distances for loads of less than 90% load factor, it must not be inferred that the many lines now in operation for transmitting steam power do not represent sound investments.

V. M. MARQUIS,[36] ESQ. (by letter).[36a]—The paper by Mr. Orrok covers the progress made in the generation of energy by heat engines in such a thorough and interesting manner that it does not leave very much to be discussed. However, there are a few points on which additional comments may be of value.

It is only necessary to compare the Newcomen engine of 1770, which required an output of 416 000 Btu per kw-hr, with the present high-pressure steam turbine unit and mercury vapor units, which require an output of only 10 000 or 11 000 Btu per kw-hr, to see the great strides that have been made in the art of generation. However, the end has still not been reached. Development of materials suitable for use at higher temperatures and pressures, the use of higher speeds, and the possible further adoption of the binary cycle, should all result in a gradual increase in the efficiency of the heat engine.

The excellent results already obtainable with the steam turbine are shown in Table 2 of Mr. Orrok's paper. Mr. Orrok is dealing with the progress that

[36] With Am. Gas & Elec. Co., New York, N. Y.

[36a] Received by the Secretary January 6, 1938.

has been made in the generation of energy by heat engines, and Table 2 most certainly emphasizes the results possible in a modern high-pressure plant. However, in any discussion with reference to a practical system, it must be remembered that all the plants are not of this vintage, and in discussing generation costs it is necessary to take into account not only the latest and most efficient plants in the system, but also the older plants. Some of the latter will have somewhat higher generation costs, but they are, nevertheless, perfectly good plants and cannot immediately be superseded by more efficient ones. They, of course, assume a different position on the load curve than the more modern plants; that is, they operate over the peak periods and normally have a very low annual load factor. On this basis they cannot be economically replaced by new capacity. In any practical system allowance must also be made for reserve capacity, and, as stated by Mr. Orrok, this has not been included in his calculations.

The generating costs are made up of operating costs and fixed charges. Operating costs may vary considerably with annual load factor. Again, if the total operating costs given in Table 2 are plotted against load factor, the result is practically a straight line. Although this would probably be nearly correct for a new modern plant with low maintenance and labor costs, and with a single boiler supplying all the necessary steam, nevertheless, in most plants—and especially in the older ones, with many turbine and boiler units—it is necessary to consider the fixed and variable portion of the operating costs. For example, there is a certain portion of the fuel charge which is a fixed cost and a certain portion which is variable. The same holds true to a certain extent with maintenance and labor. The net result is that the curve of operating costs is fairly flat at high annual load factors, but rises considerably at lower load factors. (This is definitely shown by Fig. 30 in the paper by Mr. Sporn.) Therefore, in a practical system that includes a variety of plants, from the most efficient "modern" type to those built 10 or 15 yr ago, operating costs at low annual load factors might be expected to be considerably higher than those given in Table 2.

R. L. Sackett,[37] M. Am. Soc. C. E. (by letter).[37a]—The fine review by Mr. Orrok emphasizes the contribution which the mechanical engineer has made in increasing the efficiency and size of generating units, while maintaining the capital cost per kilowatt practically constant. It is characteristic of modern civilization to continue development in one way or another. However, as the possible rate of increase in efficiency is now much reduced, it follows that the power industry must necessarily turn soon to those other factors of power cost—distribution and consumption—and give them more serious consideration than it has in the past.

Mr. Orrok's paper also calls to mind a modern problem in engineering education. One hears a great deal about the importance of "emphasizing fundamentals." The teacher of mechanical engineering has been compelled to emphasize them. When a student, the writer was required to design a Scotch

[37] Dean Emeritus of Eng., Pennsylvania State Coll., State College, Pa.

[37a] Received by the Secretary January 6, 1938.

boiler. No one asks the student to-day to design a Scotch boiler. The subjects that had to be considered as examples then were few, such as a straight-line engine, a fire-tube boiler, and a gasoline engine. To-day, the field is so complex that there is no time to go into the design of the modern boiler—or even the design of modern steam turbines, which is more important—and the teacher is compelled, in illustrating given problems, to insist more and more on the fundamental principles than on involved, time-consuming designs that after all are of specific value only in one particular field.

The writer wishes to comment also on an item which Professor Freeman mentioned but did not stress, namely, the displacement of human labor in industry as a result of reductions in the cost of power.

Americans take considerable satisfaction in the historical fact that, over a long period of years, the percentage of the population actively employed in industry has increased. However, as long as present motives continue, there is every expectation that the adoption of new labor-saving devices will also continue, and, with it, the resulting displacement of labor. The maladjustments do not concern the producer of power in large units; they are rightly the problem of the producer of goods, because it is in his plants that they occur.

That some manufacturers are already assuming the social responsibility for readjustment in such cases, is evidenced by the following incident, which came to the writer's attention some months ago: A certain rubber company was proposing to install a machine to cut out rubber soles that formerly had been cut by hand. The machine required two or three operators and would displace about forty men. The managers called in the employees and stated the facts, but said: "We are going to train you for other jobs. The installation of this machine will increase our products and the production of other departments and we will have additional jobs here and there for which you will be trained." The consequence was that there was no disturbance in that case and the employees were continued with as good or a better rate of pay.

Certainly, the responsibility for such readjustments lies, partly at least, with engineering management. Yet engineers have done very little, from an educational standpoint, about it. Where is there an educational institution within an industry or public organization whose purpose it is to assist in these readjustments? This point is to be emphasized: These men who are displaced from jobs have little facility for re-training themselves for other kinds of work.

Consequently, there are certain social influences of considerable importance that are involved in such a readjustment, and for these reasons, it seems the responsibility is even greater on engineering management to assist in this task. Certainly, engineering can do a great deal, most of which is now being done in technical institutions, night schools, or in engineering extension courses, to reach into the industries and assist these men to better jobs—or, at least, to different ones.

D. J. McCORMACK,[38] ESQ. (by letter).[38a]—A few supplementary statements may be added to the paper by Mr. Rogers on the hydro-generation of energy.

[38] Sales Mgr., S. Morgan Smith Co., York, Pa.

[38a] Received by the Secretary January 19, 1938.

Because of the relatively high cost of low-head hydro-electric plants (most of which are situated in the Middle West and along the Atlantic seaboard in the most densely populated section of the United States), and the widely variable flows usually encountered in such developments, it has been necessary to reduce all costs of such plants to a minimum to justify their development, or even the re-development of an existing plant. Hydro-electric engineers have been faced with the problem of cutting costs wherever possible to effect this purpose, particularly in the face of rising unit prices. In consequence, turbine builders have constantly been urged to develop higher-speed units. In this connection, the introduction of the Kaplan adjustable-blade turbine has made it economically possible to develop and re-develop many low-head water powers that otherwise could not have been considered. Records show that an adjustable-blade turbine plant has exceeded the kilowatt-hour output of a Francis-type turbine plant or a fixed-blade turbine plant, over the same year and on the same river, by 10 to 30 per cent.

Where close regulation is not required, electrical operation of the runner blades can be used. This method has the disadvantage of not being able to pick up load quickly to prevent a consequent drop in speed on sudden load demand. If the flow of the river varies quite gradually and shut-downs are permissible, hand-adjusted runner blades may be used at a slight saving in initial cost.

It has been determined at the Safe Harbor Plant, on the Susquehanna River, that the last unit installed, which has machine-finished blades, will safely carry a 5 000-hp greater load within cavitation limits than the other units, which do not have machined blades.

Impulse wheels, because of the higher efficiencies (88 to 90%) now obtainable therewith, and because of their simplicity of construction, durability, and desirability from an operating standpoint, are at present being adopted more generally than reaction turbines for heads of from 500 ft to 800 ft. It is recognized that because of the rapid wear of the runner seals and of the joints at the top and bottom of the gates under the higher heads, reaction turbines must be completely overhauled every few years in order to maintain efficiency. A well-designed impulse wheel will maintain high efficiency over a much longer period, and, if the tail-water variation is within a narrow range (as it generally is), an impulse wheel with a vertical setting can be placed very close to tail-water to increase the effective head.

I. E. Moultrop,[39] Esq. (by letter).[39a]—The paper by Mr. Sporn is extremely timely. His fair and comprehensive treatment of the subject of generating cost should be thoughtfully read by every one interested because it gives the complete story in terms every one can understand. For the first time the three chief items in the cost of electric energy as delivered to the customer are analyzed collectively and their relative values in terms of dollars are clearly presented.

Many people, impressed by the size of some spectacular steam or hydro-

[39] Cons. Engr., Belmont, Mass.

[39a] Received by the Secretary January 21, 1938.

electric generating station, make the erroneous deduction that the cost of such stations is the chief money outlay for the system. Reasoning from this incorrect assumption, they naturally conclude that electrical energy obtained from water, running freely down hill, must be much less costly to produce than that derived from coal, which must be bought. This would be more nearly correct if no other items were involved and the hydro-electric station was located near the load center.

The large area of the storage reservoir seldom impresses one as being an item of considerable expense. Moreover, the layman does not realize the wide range between flood and low water and the comparative duration of each, and the fact that the hydro-electric generating station must meet both extremes. Finally, in a hydro-electric development, the greater part of the money outlay is made in the initial construction; the power-house and its appurtenances are, in general, the only parts that can be built initially to just the right size to meet the demands of the existing load. Hence, considerable of the invested capital may actually be idle for a long period. This factor may be quite important, but it is seldom appreciated. The steam generating station, on the other hand, can usually be built just to meet the existing load requirements, and comparatively little capital need be tied up awaiting future demands.

The comparative costs of steam and hydro-electric generating stations, as given in the paper, are illuminating. As the cost figures are taken from Federal reports they cannot be questioned, except that one may wonder whether complete data, such as an engineer requires, are always included. Engineers familiar with both types of stations might feel that the hydro-electric station was somewhat favored.

Coming to the other two items of power cost to which the public generally gives little thought, Mr. Sporn clearly points out their greater relative importance. It is indeed unfortunate that the costs of the transmission system and of the distribution system are so little understood. If the public knew that the greater portion of the invested capital was spent in building these parts of the system there would probably be less popular agitation for hydro-electric stations.

It is significant that, on the average, a ton of coal can be transported about 140 miles at a lesser cost than that of transmitting the electrical energy produced by that ton of coal for a similar distance. If this were common knowledge, one would hear fewer arguments for developing a greater number of the remaining water-power sites—especially as these sites are usually quite undesirable and far from the load center.

Likewise, the cost of the distribution system is little realized. In the cities it is usually underground and, therefore, cannot be seen. Moreover, it is not generally appreciated that the cost of maintenance is high. Rural electrification, badly as it is needed, can be made possible only by developing a very cheap and none-to-reliable rural distribution system and by materially increasing the energy consumption per customer. Such a system could not be used in municipal areas.

Throughout the paper Mr. Sporn has been very conservative, basing many of his data on Government sources which must be accepted as correct. The curves of generating costs (Fig. 30) and of transmission costs (Figs. 31 and 32) are most illuminating and should be studied with care by all who are inclined to discuss superficially the subject of electrical energy costs.

The final paragraph, indicating future trends, is very sound and again conservative. The best engineering minds in the world in mechanical and electrical work are engaged in studies to simplify apparatus, reduce its cost, and improve its efficiency. One would be very rash to predict what the end will be, but utility engineers are looking forward to much more improvement. The 2 400-lb steam plant which Mr. Sporn now (1938) has plans for building is a decided step in advance. It is to be hoped that similar progress will be made in the transmission and distribution fields, and there is reason to expect that these hopes will be realized in due time.

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DISCUSSIONS

STREAM POLLUTION IN THE OHIO RIVER BASIN A SYMPOSIUM

Discussion

By Messrs. D. E. Davis, L. S. Morgan, C. A. Holmquist, and Robert Spurr Weston

D. E. Davis,[26] M. Am. Soc. C. E. (by letter).[26a]—The paper by Mr. Streeter supplies for the first time a quantitative measure of the effect of acid in the reduction of bacteria in the Upper Ohio River. That the "step-down" method utilized in reaching these conclusions possesses validity is demonstrated by the close correspondence between the calculated and the observed results. That acid reduces the pollution load appreciably is indicated by the *B. coli* count at Wheeling, W. Va., which under winter conditions is 25% and under summer conditions is less than 1% of what might be expected under normal alkaline conditions.

The 1932 studies of the Pennsylvania State Health Department, under the immediate direction of Mr. L. S. Morgan, District Engineer, indicated that at Sewickley, twelve miles below Pittsburgh, the results are even more striking, since under the summer low-flow conditions in 1932, the *B. coli* index was 165 (compared to 3 760 for Wheeling), or only a fractional part of 1% of the anticipated results under "normal" conditions. They also indicated that for corresponding river flows, the oxygen content of the river in 1932 was better than for the 1914–15 period, in spite of a considerable increase in the population in the District. This unexpected result may have been due to the substantial cessation in the intervening period of the activities of the brewery and distillery industries, whose wastes make a heavy oxygen demand on the rivers. The increase in acid, experienced since 1914, had very decidedly reduced the bacterial load, and on these two counts the river water was better during low-flow periods than in earlier years.

Note.—The Symposium on Stream Pollution in the Ohio River Basin was presented at the meeting of the Sanitary Engineering Division at Pittsburgh, Pa., on October 14, 1936, and published in January, 1938, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the Symposium.

[26] Engr. (The Chester Engrs.), Pittsburgh, Pa.

[26a] Received by the Secretary May 7, 1937.

Although it is known that acid reduces the "bug counts," it is not clear how this result is achieved. Whether the acid actually kills most of the *B. coli*, as chlorine does, or whether it merely makes them "play dead" like a "possum," only to come to life under alkaline conditions below, is a matter of conjecture as yet. No complete differentiation has been made as between the different strains of *coli* to lead to a certain conclusion as to whether acid may have a selective action, killing off permanently the pathogenic types and letting some other more resistant organisms continue their life cycles. An alternative possibility is that during periods of low-flow, coagulation and sedimentation effect the prompt deposition of bacteria on the river bottom, where they may lie dormant until the first freshet scours them out to increase enormously the load on down-stream filtration plants.

The rôle of plankton in the self-purification of streams under acid conditions is not yet completely understood, nor is there a definite correlation between plankton and bacteria, which might serve as a guide to the degree of purification which has taken place. It was shown in the 1914–15 studies that there was an abundance of plant life in the acid Monongahela, but that it was confined to a relatively small number of species. It was suggested that these types might have the ability of ingesting organic matter without the interposition of bacteria. It is entirely possible that some of these forms of life find optimum environments under acid conditions.

It has been noted that certain forms of grasses or "sea weeds" have begun to flourish increasingly in the Allegheny River as it has become more acid. The pools immediately below Pittsburgh (with pH usually on the acid side) are literally alive with this type of growth which flourishes on the rich river mud. The studies of the U. S. Public Health Service have shown that these plants, in generating chlorophyll by photosynthesis, give off large quantities of oxygen during daylight hours, thus measurably abetting reaeration at the surface of the water. They also exert a mechanical effect in intercepting and holding floating material, later to deposit it on the river bottom, as mud into which the plant extends its roots.

It is possible that this luxuriant growth and other plankton forms may directly break down complex organic material into simpler and more stable forms even without the aid of bacteria. In other words, under these acid conditions the absence of large numbers of the bacteria that are usually found below heavy sources of pollution may not mean that the processes of self-purification are largely postponed until more alkaline conditions obtain. The work may be taken over in large part by the river plankton.

These observations suggest the value and importance of further fundamental studies relating to the effect of acid on purification processes. They are not merely academic questions. Acid admittedly takes an economic toll in its attack on exposed steel and concrete structures, and in water treatment, but the U. S. Public Health Service has raised the question as to whether its beneficial effects on sewage may not outweigh these harmful results. A mine-sealing program is being pushed which may greatly reduce the quantity of acid entering the streams. Should this be completely successful, river conditions might become intolerable and secondary sewage treatment, at a greatly increased cost,

might become necessary in some communities. Furthermore, if acid acts as a germicide, this function should not be ignored, since it contributes to the greater safety of those using the river for recreational purposes.

These questions are susceptible of fairly definite evaluation and policy determination. The time is ripe for their early consideration so that money may not be expended wastefully. To quote the author, they "fall more especially in the field of research, which in the long run must keep pace with that of the sanitary survey, as the two are virtually inseparable in meeting engineering problems of stream-pollution control."

L. S. MORGAN,[27] ESQ. (by letter).[27a]—The Pennsylvania Department of Health during the period from September, 1932, to February, 1933, conducted a sanitary survey of the Ohio River in co-operation with the Health Departments of down-stream States and the United States Public Health Service. The survey extended over the stretch of the Ohio River from its source at Pittsburgh to the Pennsylvania-Ohio State line, a distance of approximately 44 miles. Sampling stations were located at distances of three, twelve, twenty-five, and forty-four miles below the source at Pittsburgh and at distances of six and thirteen miles, and of one mile, above the mouths of the Monongahela, Allegheny, and Beaver Rivers, respectively, which are the main tributaries of the Ohio River.

The main reason for conducting the survey was to determine the character and quality of the Ohio River water at and below the Metropolitan Pittsburgh District as a result of the discharge of untreated sewage in this metropolitan area.

The regimen of the Ohio River and its two principal tributaries, the Monongahela and Allegheny Rivers, is controlled by a series of navigation dams. These dams create large pools into which the sewage from a population of approximately 1 200 000 is discharged untreated in the Metropolitan Pittsburgh District from eighty-five separate municipalities, including the City of Pittsburgh proper.

The Monongahela and Allegheny Rivers drain large areas located in the bituminous coal fields of Southwestern Pennsylvania and West Virginia. The discharge of mine drainage from bituminous mines, together with the discharge of acid wastes from metallurgical plants, renders the water of the Monongahela River at its mouth acid practically at all times and the Allegheny River at its mouth acid especially during low-flow periods. These acids include free sulfuric acid, together with the sulfates of alumina and iron. The Ohio River, therefore, during low-flow periods contains varying quantities of the mineral acid and acid salts, the degree of acidity being dependent on the rates of flow and degrees of acidity of the two principal tributaries.

During periods of low flow and accompanying low velocity, the navigation pools in the Metropolitan Pittsburgh Area act as sedimentation basins in which are deposited settleable material contributed by the sewage and industrial waste discharges. Sedimentation is materially aided through the coagulation and precipitation of acid salts during periods of low flow. The passage of water at

[27] Dist. Engr., Penna. Dept. of Health, Greensburg, Pa.

[27a] Received by the Secretary May 7, 1937.

low velocities through the navigation pools and over navigation dams permits a high degree of surface aeration. Thus, during low-flow periods, these combined circumstances provide a method of sewage disposal at least roughly comparable to sedimentation.

However, because of the acid character of the river water, bacterial activity in the deposited sludge is inhibited and, as a result, the settled material does not undergo bacterial decomposition but remains for varying periods of time in what might be termed a "pickled" or "cold storage" condition until scoured out by sudden and rapid increases in stream discharge.

At such times, as velocities increase, the accumulated material is washed down stream in concentrated form and the sudden changes in the character of river water are reported as causing an undue burden on down-stream waterworks deriving their sources of supply from the Ohio River.

During the survey, determinations of the varying sanitary quality of the Ohio River water were obtained and correlated with varying rates of discharge (see Table 6). The station selected for securing such data was twelve miles

TABLE 6.—SANITARY QUALITY OF OHIO RIVER WATER BELOW PITTSBURGH, FOR VARIOUS RATES OF FLOW

(Data from Survey of 1932–1933)

Average daily discharge, in cubic feet per second	Dilution factors, in cubic feet per second per 1 000 population	Average dissolved oxygen, in parts per million	Average 5-day bio-chemical oxygen demand in parts per million	*Coli-Aerogenes* INDEX PER 100 CUBIC CENTIMETERS	
				Average	Maximum
1 500	1.2	6	0.2	160	700
2 900	2.4	7	0.5	160	670
7 200	6.0	11	2.5	290	3 400
19 400	16.2	12	2.6	4 300	36 700
44 000	36.6	13	3.5	5 300	37 000

below the point at Pittsburgh and below the points of discharge of practically all untreated sewage from the Metropolitan Area.

The dilution factors shown in Table 6 for the lower flows are lower than those generally recognized as necessary under ordinary circumstances to prevent nuisance conditions. In this case, the critical discharge to prevent nuisance conditions would be approximately 5 000 cu ft per sec, based on the contributing sewered population alone and disregarding entirely the equivalent population load of industrial waste pollution which, in the Pittsburgh area, would probably equal or exceed the sewered population. Nevertheless, despite the low dilution factors, general nuisance conditions do not exist in the river.

The relatively high dissolved oxygen content at the various rates of discharge indicates that the oxygen resources of the stream are not being depleted to the point where general nuisance conditions would result. These high results also indicate that the dissolved oxygen available is not being utilized for the oxidation of organic material through bacterial activity.

The five-day B.O.D. is much lower during low-flow periods than would ordinarily be anticipated, but is more nearly normal during the higher flow.

Based on the contributing sewered population the anticipated B.O.D. would probably vary from a maximum of approximately 37 ppm at the low flows to a minimum of approximately 1.5 ppm at the high flows, with the organic pollutional load uniformly distributed over the entire flow range. Assuming a reduction in organic material of 35%, equivalent to the removal obtained through sedimentation during low-flow periods, the resulting anticipated oxygen demand would still greatly exceed the observed values. It is, therefore, believed that the concentration of acid during low-flow periods renders the B.O.D. determination unsatisfactory as a yardstick for measuring the true pollutional load of organic material and that some revision in the standard procedure is necessary to obtain a true picture under such conditions.

The *coli-aerogenes* index shows clearly the inhibiting effect of the acid condition in the river at low stages. With increased discharge, the index shows a decided increase probably more nearly representing normal conditions. In the absence of acid wastes and under similar pollutional loads much higher indices would be anticipated.

These results indicate a fairly stable condition in the Ohio River below Pittsburgh during periods of low-stream flow due to the presence of acid wastes inhibiting normal biological and chemical processes of purification and preventing general nuisance conditions. With increasing stream flow, however, resulting in more normal conditions through dilution of the acid wastes, the results more nearly represent the anticipated conditions resulting from the discharge of untreated sewage from such a large population.

C. A. Holmquist,[28] Esq. (by letter).[28a]—In reviewing Mr. Stevenson's paper the New York State Department of Health finds itself in general agreement with the treatment which he has given the subject. During the July, 1935, floods, conditions in New York State were comparable with those suffered in Pennsylvania and other States during March, 1936, and the Health Department's activities and policies with respect to protection of the public health at that time closely paralleled those undertaken and followed in Pennsylvania. During the 1936 flood, conditions were less severe in New York State. However, these two flood experiences have enabled the New York State Department of Health to formulate a somewhat definite plan of action and policy to be followed in future floods.

There is one point mentioned in Mr. Stevenson's paper however, with which the writer is not in complete agreement. This is in reference to the issuance of general and unqualified "boil-water" warnings in times of such catastrophes. There are, of course, certain differences in the organization of public health work in the two States that might make the policy followed by Mr. Stevenson perfectly proper for his State and improper for New York. In New York State there are many competent local health officers and water-works officials upon whom the Health Department can always rely—who would be the first to advise the Department if anything were wrong—and upon whom it would place the responsibility for deciding whether or not a "boil-water" order should be

[28] Director, Div. of Sanitation, New York State Dept. of Health, Albany, N. Y.

[28a] Received by the Secretary May 7, 1937.

issued. During both the 1935 and 1936 floods the Department issued promptly general "boil-water" warnings, and it is believed that such action is advisable. However, these warnings were qualified in such a way that they left to the local health officer final decision in the matter. People were advised generally to be guided by the suggestions of their local health officers and water-works officials, but, in case there was any doubt as to the safety of the supply, to boil their water or treat it with chlorine compounds of various kinds as a precautionary measure, until the exact condition of the supply could be ascertained. Had these general warnings been issued without some qualifications, the people of Albany, for example, would have had to boil their water—which would have been unfortunate in view of the known safety of that supply. Even with the qualified "boil-water" warnings there were several instances when health officers issued local warnings which were unnecessary. However, in such cases, the local health officers were uncertain as to the conditions, and there is no criticism under the conditions prevailing of "erring on the side of safety."

Mr. Stevenson has stated that the primary responsibility for protecting the public health in time of floods should rest upon local authorities. This should be emphasized in the strongest terms possible. State Health Departments and other units of State Government can furnish valuable guidance and assistance during emergencies, but local health officers, local water-works officials, and other local officials have the real opportunity of reducing to the irreducible minimum the public health hazards which invariably arise during such periods. The protection to the public health at such times is likely to be very much in proportion to the ability of the local officials to meet the problems promptly and properly.

Therefore, in any planning which should be done in advance to meet the public health needs of possible future catastrophes, training of local health officers and water-works operators or superintendents should be the central objective. A State Health Department should strive to provide continuously, through water-works schools, frequent inspections of supplies, etc., for the general education and training of local officials. It should seek to bring all of them to a high level of competence so that when floods occur they will know what to do, how to do it, and have the essential equipment at hand for their work.

In stressing the importance of the training of local health officers and water-works operators, the writer is not unmindful of the great need for physical improvements in supply systems that will lessen the danger of failure or contamination during floods. Water-works officials should throw overboard the philosophy of "The Arkansas Traveller," who "wouldn't repair the leak in the roof when it was raining, because then he couldn't work, or when it was not raining, because then it didn't leak." The time to make improvements to lessen the hazards to public water supplies occasioned by floods is before the floods occur. State health officials frequently come to the end of their rope in appealing and preaching to the governing boards of municipalities for favorable action on important matters. The chance of success is always enhanced considerably when there exists in the local community a health officer or local water-works man who is thoroughly trained and imbued with the proper sense of public health

values. Therefore, the proper training of local officials is the important objective. Carried to the proper end effort expended in this direction will not only equip the local health officer or water-works official to respond with the maximum efficiency during any emergency but will also help to bring about many of the important improvements in public water supplies which will lessen the dangers during emergencies and which to some extent may at present be opposed by insufficiently impressed local officials.

ROBERT SPURR WESTON,[29] M. AM. SOC. C. E. (by letter).[29a]—In Mr. Stevenson's thorough review of the sanitary problems incidental to floods, he has set down little to criticize and much to commend. His administrative experience as public health engineer for a large and varied State permits him to speak with authority. The statement of a private practitioner must necessarily be limited in comparison.

More people comprehend the practical engineering problems in the construction of flood reservoirs than the social problems involved. Imagine, for example, as might be the case on more than one river system, a series of upland valleys which are excellent sites for flood control reservoirs but which are occupied by expensive summer residences, country clubs, and the hamlets which they support. Down below are mills and cities, damageable by floods and supporting a large population; but try to apply to the situation the methods so successful on the Miami River, and then watch the summer residents rush to the State Houses and to their powerful political friends. They would say "dike or otherwise protect," or, "move low-class dwellings and shops away but leave us our homes and our æsthetic enjoyment." Again, there is the resistance of the taxpayer to benefits which accrue to riparian dwellers but not directly to him. These and other difficulties are often controlling.

It is reported that when Germany decided to war against France in 1870, von Moltke took a stack of telegrams from his desk and, sending them, mobilized the Prussian Army for victory. An analogous preparation is required for the successful control of the public health during floods, as Mr. Stevenson has shown.

As he states, the safeguarding of water supplies is of greatest importance. It is far better, however, to have water-works flood-proof than to depend upon emergency measures when the floods arrive.

In Massachusetts, when the laboratory of the Department of Public Health, at Lawrence, was flooded into temporary desuetude, a number of private laboratories were called upon to assist. Viewing their results, it was rather surprising that even when the flood was at its height almost none of the tube well supplies in the flooded valleys went bad. On one day, out of eighteen samples collected, mostly from ground-waters in the flooded Charles River Valley, only one was polluted and that one only slightly, showing the value of good construction and the protection given by impervious layers in the soil.

It must be realized, however, that Massachusetts is particularly fortunate in having only one water supply from a polluted river. The problems which

[29] Cons. Engr. (Weston & Sampson), Boston, Mass.

[29a] Received by the Secretary May 7, 1937.

confront the cities whose sites dot the courses of rivers in the great valleys, where rivers are necessarily both sources of water and places of final disposal of sewage and waste, are happily absent. In Massachusetts, too, 97% of the population has access to public water supplies, and 87% is tributary to sewerage systems.

During the flood in Massachusetts, the inspiring aspect was the efficiency of the Department of Public Health; the depressing aspect was the lack of comprehension and organization shown by some of the municipal departments. Co-operation among towns under State leadership was excellent. At Lawrence, for example, hose connections were made to the hydrants of three neighboring towns and the supply was thus maintained.

Flood control is important, but safe water supplies and sewage disposal are more important. Consequently, no scheme of control should be planned without regard to the public health factors, and the judgment of the health authorities should be taken into account in any decision regarding works for flood control.

The U. S. Supreme Court, through the late Associate Justice Oliver W. Holmes, lucidly stated that "a river is not an ordinary amenity but a treasure." If so, all the treasurers, including the hydrologist and the public health engineer, should control the assets and liabilities. Furthermore, it seems best to have these treasurers appointed by the States and do the business which is necessarily inter-state by using compacts and agreements, rather than by submitting to Federal regulation or direction. Federal data and advice, however, should be welcomed and their costs cheerfully borne.